# PARTIAL REPLACEMENT OF ASPHALT BITUMEN WITH ANYWAY NATURAL SOIL STABILIZER

**BY**

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# DECLARATION

I certify that research work titled "***Partial replacement of Asphalt Bitumen with ANSS (Anyways Natural Soil Stabilizer)***" is my own work. The work has not been presented elsewhere for assessment and award of any degree or diploma. Where material has been used from other sources it has been properly acknowledged / referred.

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## APPROVAL PAGE

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## ABSTRACT

*Asphalt (bitumen) is used in road pavement as a binder of aggregate in great extent all over the word. Asphalt pavement must undergo heavy loads and unfavorable environment condition for unacceptable period of time. High temperature rutting and low temperature cracking are the most considerable limitation of unmodified and pure asphalt modification and reinforcement of asphalt binder is necessary.*

*Therefore, the objective of this thesis is to replace partially asphalt binder with ANSS and evaluate the high temperature performance of asphalt binder modified with anyway natural soil stabilizer (ANSS). i.e., to examine the contribution of this modified binder to minimize rutting.*

*ANSS commonly composed of a specific type of cement, a lime, pozzolans, and rate governing additives, and a unique polypropylene fiber other name poly (propene) used as concert additives to increase strength and reduce cracking. The study compares the rheological and empirical properties of neat asphalt binder and asphalt binder containing ANSS. In addition to the control specimen, four binders were obtained by mixing the asphalt binder with three different percentages of ANSS by weight of asphalt binder (i.e. 3%, 6%, 9% and 12%). Empirical tests such as penetration, ductility, softening point and Flash and Fire point were conducted. Four rheological tests were conducted namely Amplitude Sweep Test (AST), Frequency Sweep Test (FST), Performance Grade and Multi-Stress Creep Recovery (MSCR) using Marvel Bohlin instrument. The bitumen binders were aged by rolling thin film oven (RTFOT) to simulate short-term aging. AST and FST were conducted for both unaged and aged samples at temperatures of $10^0$c ,21.1°c, 37.8°c, and 54.4 °c. Performance grade were performed for both unaged and aged samples. Whereas, the MSCR test were conducted on aged samples only at temperatures of 52°c, 58°c & 64°c.*

*The results showed that the addition of ANSS has affected the properties bitumen binder positively as illustrated in empirical tests. in rheological tests ANSS modified binder has improved rheological properties of binder at higher temperature with higher complex modulus and lower phase angle. The rutting performance improved as depicted from MSCR test having lower total strain, lower non-recoverable creep compliance (Jnr) for ANSS modified binders. In conclusion, addition of ANSS improves the rheological* *properties, aging effect and rutting performance of binders at high temperature ranges, as a result it might be used in areas where rutting is more critical.*

*Keywords: ANSS, Rheological properties, Aging effect, Rutting performance, Binder*

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# ACRONYMS

| | |
|---|---|
| AASHTO | American Association of State Highway and Transportation Officials |
| AACRA | Addis Ababa City Road Administration |
| AST | Amplitude Sweep Test |
| ANSS | Anyway Natural Soil Stabilizer |
| CRBC | China Road and Bridge Corporation |
| DSR | Dynamic Shear Rheometer |
| ERA | Ethiopian Road Authority |
| ERCC | Ethiopia Road Construction Corporation |
| EVA | Ethylene Vinyl Acetate |
| FST | Frequency Sweep Test |
| FHWA | Federal Highway Way Administration |
| HMA | Hot Mix Asphalt |
| IFH | International First highway |
| LVER | Linear Viscoelastic Range |
| MSCR | Multiple Stress Creep and Recovery |
| PAV | Pressure Aging Vessel |
| PVC | Palatizes polyvinyl chloride |
| PG | Performance Grade |
| PR | Percent Recovery |
| RTFO | Rolling Thin Film Oven |
| RV | Rotational Viscometer |
| SBA | Styrene butadiene-styrene |
| SHRP | Strategic Highway Research Program |

| | |
|---|---|
| SUPERPAVE | Superior Performance Pavement |

# TABLE OF CONTENTS

DECLARATION....ii
APPROVAL PAGE.... iii
ABSTRACT ....iv
ACKNOWLEDGMENTS ....vi
Acronyms....vii
Table of Contents....ix
List of Figures.... xiii
List of Tables ....xv
Chapter 1 INTRODUCTION ....1
1.1 Background of the study ....1
1.2 Problem Statement ....2
1.3 Objective of the Study....3
1.3.1 General objective of the study ....3
1.3.2 Specific objective of the study....3
1.4 Limitation....3
1.5 Organization....4
Chapter 2 Literature Review ....5
2.1 Introduction....5
2.2 Bitumen Fundamentals ....5
2.3 Composition of the Bituminous Materials ....6
2.4 Behavior of Bitumen....6
2.4.1 High Temperature Behavior ....7
2.4.2 Low Temperature Behavior....7
2.4.3 Intermediate Temperature Behavior ....7
2.5 Asphalt Binder Characteristics....8
2.6 Asphalt Rheology....9

2.7 Visco-Elastic Characterization of Bitumen........................................................10
2.8 The working principle of DSR..........................................................................12
2.9 Bitumen Ageing.................................................................................................16
2.10 Bitumen Modification ......................................................................................17
2.11 Benefits of bitumen modification....................................................................18
2.12 Asphalt Additives..............................................................................................20
2.13 The Need of asphalt additives ..........................................................................20
2.14 Historical Background of Anyways Natural Soil Stabilizer (ANSS).............21
*2.15* Uses of ANSS in road Construction.................................................................21
2.16 Effect of Lime on asphalt ..................................................................................22
2.17 Rheological Data Presentation ..........................................................................22
2.18 Summary ...........................................................................................................24
Chapter 3 METHODOLOGY ...................................................................................25
3.1 Introduction...........................................................................................................25
3.2 Materials...............................................................................................................25
3.3 Experimental Plan.................................................................................................25
3.4 Conventional Tests................................................................................................26
3.4.1 Penetration Test ................................................................................................26
3.4.2 Softening Point .................................................................................................27
3.4.3 Ductility ............................................................................................................28
3.4.4 Fire and flash point-..........................................................................................28
3.5 Dynamic Shear Rheometer (DSR) Tests ...........................................................29
3.5.1 Sample Preparation for Rheological Test .......................................................29
3.5.2 Basic Test Procedure ........................................................................................30
3.5.3 Tests on the DSR ...............................................................................................31
3.6 Performance Grade Determination ....................................................................34
3.7 Test Temperature and Work Plan .......................................................................35

3.7.1 Test temperature ..... 35

3.7.2 Work Plan ..... 36

Chapter 4 Result and analysis ..... 39

4.1 Introduction ..... 39

4.2 Effect of ANSS on Asphalt Binder ..... 39

4.2.1 Effect of ANSS on Penetration ..... 39

4.2.2 Effect of ANSS on Softening point ..... 40

4.2.3 Effect of ANSS on Ductility ..... 40

4.2.4 Effect of ANSS on Fire and Flash Point ..... 40

4.2.5 The Effect of ANSS on Amplitude Sweep Test ..... 41

4.3 Black Space Diagram ..... 43

4.4 The Effect of ANSS on Frequency Sweep Test ..... 44

4.5 Master Curve ..... 45

4.1 Phase Angle Master Curve ..... 50

4.2 Statistical Analysis of FST Result Using ANOVA ..... 51

4.3 Summary ..... 57

CHAPTER 5: CONCLUSION AND RECOMMENDATION ..... 58

5.1. Conclusion ..... 58

5.2. Recommendation ..... 59

5.3. Future Study ..... 59

REFERENCES ..... 60

Appendix-A Conventional Test Result ..... 63

Appendix-B Effect of Temperature of AST ..... 66

Appendix-C Frequency Sweep Test result ..... 71

Appendix-D Statistical Analysis Using ANOVA ..... 76

Appendix E - Multi Stress Creep and Recovery Test Results ..... 81

Appendix F- Sample photos ..... 97

## LIST OF FIGURES

Figure 2-1. Visco-Elastic **Behavior of Asphalt [18].** ........................................................ 7
Figure 2-2. Viscous and Elastic Behavior **[18].** ................................................................ 11
Figure 2-3. Dynamic Shear Rheometer Geometry **[18].** .................................................. 12
Figure 2-4. DSR Geometric Parameters **[18]** .................................................................. 13
Figure 2-5. Stress-Strain Output for a constant Stress Rheometer **[18].** .......................... 13
Figure 2-6. Stress-Strain Response of a Viscoelastic Material **[18].** ................................ 14
Figure 2-8. Deformation Due to Number of Loads **[6].** ................................................... 16
Figure 3-1. Experimental Flow Chart .................................................................................. 26
Figure 3-2. Experimental setup of Penetration Test ........................................................... 27
Figure 3-3. Experimental setup of Softening Point Test ..................................................... 28
Figure 3-4. Experimental setup of Ductility Test ............................................................... 28
Figure 3-5. Experimental Setup of Flash and Fire Point Test ............................................ 29
Figure 3-6. Specimen Prepared for DSR Test .................................................................... 30
Figure 3-7. Dynamic Shear Rheometer Setup .................................................................... 30
Figure 3-8. Amplitude Sweep to Determine Visco-Elastic Region **[7]** ............................ 33
Figure 4-1. Penetration Test Result ..................................................................................... 39
Figure 4-2. Effect of ANSS on Softening Point ................................................................. 40
Figure 4-3. Effect of ANSS on Softening Point ................................................................. 41
Figure 4-4. Linear Visco Elastic Range for 6 % ANSS before RTFO ............................... 42
Figure 4-5. Black Space Diagram for Binder Mixes .......................................................... 44
Figure 4-6. Complex modulus verses frequency for aged 6% ANSS ................................ 45
Figure B-1. Effect of temperature on original Binder ........................................................ 66
Figure B-2. Effect of Temperature on 3 % ANSS Unaged Binder .................................... 66
Figure B-3. Effect of Temperature on 6 % ANSS Unaged Binder .................................... 67
Figure B-4. Effect of Temperature on 9 % ANSS Unaged Bin .......................................... 67
Figure B-5. Effect of Temperature on 12 % ANSS Unaged Binder .................................. 68
Figure B-6. Effect of temperature on 0 % Aged Binder ..................................................... 68
Figure B-7. Effect of temperature on 3 % Aged Binder ..................................................... 69
Figure B-8. Effect of temperature on 6% Aged Binder ...................................................... 69
Figure B-9. Effect of temperature on 9% Aged Binder ...................................................... 70
Figure B-10. Effect of temperature on 12 % Aged Binder ................................................. 70
Figure C-1. Unaged neat Binder ......................................................................................... 71

Figure C-2. aged neat Binder ........ 71
Figure C-3. 3% Unaged Binder ........ 72
Figure C-4. 3 % aged Binder ........ 72
Figure C-5. 6 % Unaged Binder ........ 73
Figure C-6. 6 % aged Binder ........ 73
Figure C-7. 6% Unaged Binder ........ 74
Figure C-8. 9 % aged Binder ........ 74
Figure C-9. 12 % Unaged Binder ........ 75
Figure C-10. 12 % aged neat Binder ........ 75

# LIST OF TABLES

Table 1. Set of Binder Test According to Superpave ....................................................35
Table 2. Experimental Work Plan ....................................................................................36
Table 3. Shift factor for Complex Modulus Master Curves for aged and Unaged Binder ............................................................................................................................47
Table 4. Summary of a hypothesis test of Master Curve ................................................52
Table 5. MSCR Test Temperature Based on PG ............................................................53
Table 6. Summery of Jnr value for different percentage of ANSS .................................55
Table 7. Binder Specification Requirement Based on MSCR test .................................55
Table 8. Empirical test of virgin and modified binders ...................................................63
Table 9. Statistical analysis for FST at f = 0.1 Hz using ANOVA .................................76
Table 10. Statistical analysis for FST at f = 10 Hz using ANOVA .................................76
Table 11. Statistical analysis for FST at f = 25 Hz using ANOVA .................................77
Table 12. Performance Grade Determination for neat Unaged asphalt binder ................78
Table 13 Performance Grade Determination for 3 % Unaged asphalt binder .................78
Table 14 Performance Grade Determination for 6 % Unaged asphalt binder .................78
Table 15. Performance Grade Determination for 9 % Unaged asphalt binder ................78
Table 16. Performance Grade Determination for 12 % Unaged asphalt binder ..............79
Table 17. Performance Grade Determination for neat RTFO Aged asphalt binder .........79
Table 18. Performance Grade Determination for 3 % RTFO Aged asphalt binder .........79
Table 19. Performance Grade Determination for 6 % RTFO Aged asphalt binder .........79
Table 20. Performance Grade Determination for 9 % RTFO Aged asphalt binder .........80
Table 21. Performance Grade Determination for 12 % RTFO Aged asphalt binder .......80

# CHAPTER 1 INTRODUCTION

## 1.1 Background of the study

Roads make a crucial contribution to economic development and growth and bring important social benefits. They are of vital importance in order to make a nation grow and develop. In addition, providing access to employment, social, health and education services makes a road network crucial in fighting against poverty. Roads open up more areas and stimulate economic and social development. For those reasons, road infrastructure is the most important of all public assets.

The rheological properties of bituminous binders govern the subsequent performance of special hot mixtures in pavements. In the case of road pavements, the constraints give rise to moving vehicles which are of dynamic origin, and dynamic rheology can be used to analyze the visco-elastic behavior of the materials subject to loadings whose frequencies are close to those to which the road is subjected.

In general, road pavement performance properties are mainly affected by the bitumen binder properties; it is well known that the rheological properties and durability of conventional bitumen are not sufficient to resist pavement distresses ***[5].*** Therefore, asphalt researchers looking for different types of bitumen with excellent rheological properties, which directly affect asphalt pavement performance ***[3].***

Highway engineer must consider user requirement of safety and economy. As an Effect, roads should serve for its design period with minimum maintenance.

Most of the roads in Ethiopia are flexible pavement type. Flexible pavement typically consists of asphalt mixture placed over granular base layer supported by the compacted soil, referred to as the subgrade. Flexible pavement structure consists of subgrade, sub base, base course and surface course. The surface course is the upper layer which is directly in contact with traffic load. It is made of asphalt concrete which consists of high quality and expensive materials compared to other materials in other layers.

The increase in energy cost, need for improvement of pavement quality and strong worldwide demand for petroleum as well as concern over pollution and climate change has encouraged the researcher for the development of alternative binders to modify or totally replace asphalt binder.

One of these replacement alternatives might be Anyway natural soil stabilizer (ANSS) It is calcium driven, inorganic soil stabilizer patented worldwide also cost-Effective method of converting poor quality soil into a strong impermeable layer. It offers significant savings in the construction of pavement layers, embankments and reinforced earth structures, also in areas where they were not previously economically viable, minimizing the project's Effect on the environment ***[17].***

The main components that are used to formulate ANSS are a series of inorganic hydration activated powders. It is composed of a specific type of cement, a lime, several pozzolans, rate governing additives, and a unique polypropylene fiber other name poly (propene) (used as concert additives to increase strength and reduce cracking). The specific formulation allows for the individuality of the components to contribute to the reaction process, but also act holistically contributing of the stabilization process.

The theory behind their reactivity is quite simple, but the chemistry of each individual powder differs and the collaborative reaction is quite complex. Each component reacts individually while also contributing to the broader stabilization reaction. Each component contained in the stabilizer has its own series of reactions that occur at varying rates, which can be broken down into initial, short term and long-term reactions ***[17].***

Since asphalt binder is imported and expensive, a substitute that would provide a satisfactory road surface, a suitable performance, economical and at the same time make use of inorganic hydration activated powders worthy of consideration. The purpose of this research is to experiment and examine ANSS as an alternative partial replacer for asphalt binder to create a paving material for road construction.

## 1.2 Problem Statement

As a developing country, Ethiopian government allocates most of the budget for infrastructure construction. and, from these road construction and maintenance asphalt binder takes large portion of the budget. Hence, it is important to go for careful evaluation of cost-effective asphalt binder as an alternative. As a result, the use of partial use of ANSS for asphalt binder in HMA might be a possible solution.

## 1.3 Objective of the Study

### 1.3.1 General objective of the study

The general objective of the study to evaluate the Effect of ANSS modified bitumen on asphalt performance which is less susceptible to permanent deformation and improve other properties of the asphalt mixture.

### 1.3.2 Specific objective of the study

The specific objectives of this research are:

- To study conventional properties of the asphalt binder containing ANSS.
- To analyze rheological properties of the binders.
- To analyze permanent deformation property of binder containing ANSS.

## 1.4 Limitation

For DSR tests there is no complete facility to carry out the laboratory works at full scale. Because of this, during the experimental works there were some limitations especially related to sample preparation.

- To blend the modifier with the virgin bitumen, there was no convenient heater to maintain the mixing temperature. Simple common hot plate was used by trying to maintain the temperature from 160$^{o}$c to 170$^{o}$c.
- The stirrer used was homemade.
- In view of latest equipment available recently, the DSR used is old equipment and it may have associated limitations like stress resolution and other precisions.
- Because of the absence of Pressure Aging Vessel (PAV), Bending Beam Rheometer (BBR) and Direct Tension Test (DTT), low temperature binder characterization was not conducted for PG determination.

## 1.5 Organization

Chapter one defines the overall importance of the problem areas and provides an introduction into what the research is all about, chapter two deals with literatures on basic pavement concepts of pavement materials and past studies and works on pavements ANSS as a construction material. Chapter three describes how the experimental work is done with detailed procedures and the Effects are analyzed and discussed in chapter four Conclusions derived from experimental Effects and recommendations for this study and other further studies are presented in chapter five.

# CHAPTER 2 LITERATURE REVIEW

## 2.1 Introduction

This time in Ethiopia one of the active industries is road construction. When we consider the paved roads almost all are flexible pavements with asphalt concrete surfacing. The common problems in flexible pavements are pavement distresses which usually need continuous effort to tackle them. There are several ways to improve performance of asphalt concrete pavements. Some of the methods are improving the pavements material mix design, improving the construction methods, enhancing maintenance techniques and producing a new binder with improved physical, chemical and rheological properties [1]. According to the Federal Highway Administration (FHWA), the asphalt binder will affect the various performance aspects of the asphalt mixture such as permanent deformation, fatigue cracking, and low temperature cracking. The Superpave binder specification is intended to select the binder to optimize its effect on the performance of the pavement. The binder is selected based on the climate of the pavement where it will be used, the expected traffic, and the location in the pavement structure. The binders are evaluated at the expected highest pavement temperature and lowest pavement temperatures [1].

One of the most important solutions for pavement distress is to develop a new binder with the help of an additive. And if the binder selection and specification has to be considering the climate where the road way exists, it is necessary to produce a modified binder for every locality by knowing its pavement temperature.

Considering major pavement distresses in Ethiopia, the concern of this study is to evaluate the rheological characteristics of unmodified binder and binder modified with ANSS. The evaluation focuses on high temperature property of binder which will be done with the help of fundamental or Dynamic Shear Rheometer (DSR) tests.

## 2.2 Bitumen Fundamentals

Asphalt binder is defined by the American Society for Testing and Materials (ASTM) as a dark brown to black cementitious material in which the predominating constituents are bitumen that occur in nature or are obtained in petroleum processing. In the crude oil refineries, the cementitious material is in the bottom of the vacuum distillation columns. The residue of this vacuum distillation is then known as steam refined asphalt cement. As cement, asphalt is especially valuable to the pavement applications because it is strong, readily adhesive, highly waterproof, and durable. It provides limited flexibility to mixtures of mineral aggregates. It is also highly resistant to the reaction with most acids, alkalis, and salts. ***[8].***

## 2.3 Composition of the Bituminous Materials

The chemical composition of bitumen consists of different fractions, known as SARA fractions, (saturates, aromatics, resins and asphaltenes) [15]. These fractions are grouped in to Asphaltenes and Maltenes. Maltenes is further classified as saturates, aromatics and resins. Asphaltenes are known to be the insoluble part of asphalt in n-heptane, and the other group (saturates, aromatics and resins) together represent the soluble part of asphalt in n-heptane. And the elementary analysis of asphalt contains carbon (80 - 88%), hydrogen (8 - 11%), Sulphur (0 - 6%), oxygen (0 - 1.5%), and (0 - 1%) nitrogen ***[15].***

## 2.4 Behavior of Bitumen

The most mysterious property of an asphalt binder is its temperature susceptibility which makes it desirable and tricky at the same time. i.e., its measured properties are very dependent on its temperature. Asphalt cement is sometimes referred to as a visco-elastic material because it simultaneously displays both viscous and elastic characteristics. At high temperatures, asphalt cement acts almost as a viscous fluid. In other words, when heated to a high enough temperature (e.g., > $100^0$C), it displays the consistency of a lubricating fluid such as motor oil. At very low temperatures (e.g., < $0^o$C), asphalt cement behaves mostly like an elastic solid. i.e., it acts like a rubber band. When loaded it stretches or compresses to a different shape. When unloaded, it easily returns to its original shape. At intermediate temperatures, which also happen to be those in which pavements are expected to function, asphalt cement has characteristics of both a viscous fluid and an elastic solid ***[22].***

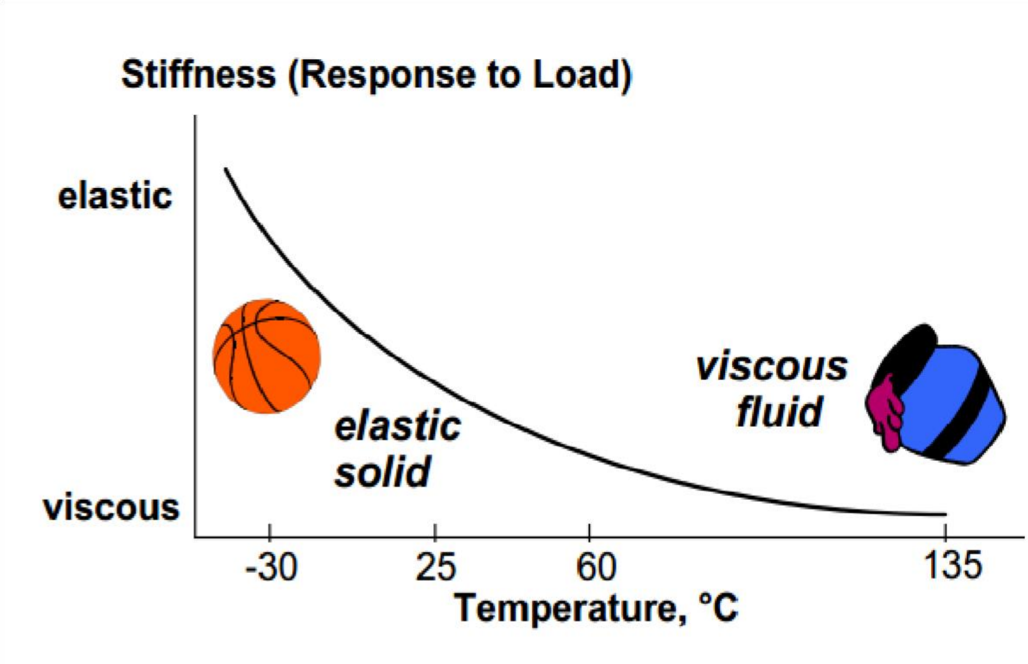



*Figure 2-1. Visco-Elastic* ***Behavior of Asphalt [18].***

### 2.4.1 High Temperature Behavior

At high temperatures (e.g., desert climate) or under sustained loads (e.g., slow moving trucks), asphalts cements act like viscous liquids and flow. This viscous behavior is characterized by its Viscosity, which is the material characteristic, used to describe the resistance of liquids to flow ***[21].***

### 2.4.2 Low Temperature Behavior

In cold climates (e.g., winter days) or under rapid loading (e.g., fast moving trucks), asphalt cement behaves like an elastic solid. Elastic solids are like rubber bands; when loaded they deform, and when unloaded, they return to their original shape. If too much load is applied, elastic solids may break. Even though asphalt is an elastic solid at low temperatures, it may become too brittle and crack when excessively loaded. This is the reason low temperature cracking sometimes occurs in asphalt pavements during cold weather. In these cases, loads are applied by internal stresses that accumulate in the pavement when it tries to shrink and is restrained ***[21].***

### 2.4.3 Intermediate Temperature Behavior

Most environmental conditions lie between the extreme hot and cold situations. In these climates, asphalt binders exhibit the characteristics of both viscous liquids and elastic solids. Because of this range of behavior, asphalt is an excellent adhesive material to use in paving, but an extremely complicated material to understand and explain. When heated, asphalt acts as a lubricant, allowing the aggregate to be mixed, coated, and tightly-compacted to form a smooth, dense surface. After cooling, the asphalt acts as the glue to hold the aggregate together in a solid matrix. In this finished state, the behavior of the asphalt is termed viscoelastic; it has both elastic and viscous characteristics, depending on the temperature and rate of loading ***[21].***

## 2.5 Asphalt Binder Characteristics

Asphalt binder has the following five characteristic properties.

- Adhesion

Bitumen has excellent adhesive qualities provided the conditions are favorable. However, in the presence of water the adhesion does create some problems. Most of the aggregates used in road construction possess a weak negative charge on the surface. The bitumen aggregate bond is because of a weak dispersion force. Water is highly polar and hence it gets strongly attached to the aggregate displacing the bituminous coating.

- Elasticity

When one takes a thread of an asphalt binder from a sample and stretches or elongates it, it has the ability to return to a length close to its original length eventually. This property is referred to as the elastic character of bitumen.

- Plasticity

When temperatures are raised, as well as when a load is applied to bitumen, the bitumen will flow, but will not return to its original position when load is removed. This condition is referred to as plastic behavior.

- Visco-elasticity

Asphalt binder has a Viscoelastic character. Its behavior may be either viscous or elastic depending on the temperature or the load it is carrying. At higher temperatures and slow loading condition there is more flow or plastic behavior, while at a lower temperature, bitumen tends to be stiff and elastic. At intermediate temperatures it tends to be a combination of the two.

- ❖ Aging

Aging refers to changes in the properties of asphalt binder over time, which is caused by external condition. There are two stages of a pavement's life where oxidation can occur in the field.

- ❖ Hot mixing and construction: During the mixing and placement process the asphalt binder is exposed to elevated temperatures and a large contact area with the aggregates which can lead to rapid aging by volatilization and oxidation. The aging mechanism which includes the loss of volatiles and chemical oxidation that result from elevated mixing and placement temperatures falls under the primary process which is followed by oxidation in a secondary process during long term service.
- ❖ In-service: The constituent asphalt binder slowly ages as the oxygen from the surrounding environment percolates through the HMA and chemically reacts during the life of an in-service HMA pavement [23].

## 2.6 Asphalt Rheology

Rheology, by definition, is the study of the flow and deformation of matter under the influence of an applied stress. Regarding the asphalt binder, the response to a stress is both dependent on temperature and loading time and consequently the rheology of asphalt binder can be expressed by its stress-strain-time-temperature response.

Asphalt binders deform when subjected to loads and their properties also change with varying temperatures. The deformation is a combination of elastic response and viscous flow [19]. (The magnitude of deformation, or mechanical response, is dependent on load magnitude, duration, and rate of application and the temperature state of the material [19]. Since asphalt binders display both elastic and viscous response properties, they are classified as viscoelastic materials. An elastic material experiences recoverable deformation when subjected to a constant load and will immediately deform and maintain a constant strain when loaded. Also, the material will immediately return to its initial shape when the creep load is removed. A viscous Newtonian material, when subjected to a constant load, will deform at a constant rate until the load is removed. The deformation of the viscous material, however, will remain after the load is removed; hence, a viscous material experiences non-recoverable deformation.

A viscoelastic material, when subjected to a creep load, experiences an immediate deformation followed by a continued time-dependent deformation ***[27].*** The immediate deformation corresponds to the material's elastic response and the time-dependent deformation corresponds to the material's viscous response. Once the load is removed, the viscous deformation component immediately ceases, but this deformation is not recovered. The delayed elastic deformation component is slowly recovered at a decreasing rate. Thus, a viscoelastic material experiences only a partial recovery of the deformation resulting from creep loading ***[27].*** The viscoelastic behavior of asphalt can be characterized by its deformation resistance and the relative distribution of that resistance between the elastic component and the viscous component within the linear range ***[27].*** The relative distribution of the resistance between the elastic component and the viscous component is dependent on the asphalt cement characteristics and temperature and loading rate. The previous loading-response descriptions are for responses within the linear range, which is characterized by the deformation being directly proportional to the applied load at any time and temperature. Nonlinear loading responses are difficult to model for viscoelastic materials such as asphalt. Linear response models, however, are sufficient for the engineering analysis of asphalt binder response to the loading conditions and environmental stresses encountered in the field.

## 2.7 Visco-Elastic Characterization of Bitumen

The visco-elastic nature of asphalt binder varies with the variation in temperature which requires to be characterized with the best technology available. Visco-elastic means that it simultaneously shows the behavior of an elastic material (e.g. rubber band) and a viscous material (e.g. Molasses). The relationship between these two properties is used to measure the ability of the binder to resist permanent deformation and fatigue cracking. To resist rutting, a binder needs to be stiff and elastic; to resist fatigue cracking, the binder needs to be flexible and elastic. The balance between these two needs is a critical one.

The Dynamic Shear Rheometer (DSR) is used to characterize the viscous and elastic behavior of asphalt binders. It does this by measuring the complex shear modulus (G*) and phase angle (δ) of asphalt binders. G* is a measure of the total resistance of a material to deforming when repeatedly sheared. Delta (δ) is an indicator of the relative amounts of recoverable and non-recoverable deformation. The value of G* (G star) and δ (delta) for asphalts are highly dependent on the temperature and frequency of loading. At high temperatures (well above pavement temperatures) asphalts behave like viscous fluids as indicated by the vertical arrow and at very low temperatures (well below pavement temperatures) asphalts behave like elastic solids as indicated by the horizontal arrow below in figure 2.

At temperatures where most pavements carry traffic, asphalts (like those represented by arrows 1 and 2) simultaneously act like viscous liquids and elastic solids. When loaded, part of the deformation is elastic (recoverable) and part is viscous (non-recoverable). That is the reason why asphalt is called a visco-elastic material. For example, even though both asphalts in Figure-2 visco-elastic, asphalt is more elastic than asphalt 1 because of its smaller δ ($\delta_2 < \delta_1$) while $G^*_1$ & $G^*_2$ are equal in value.

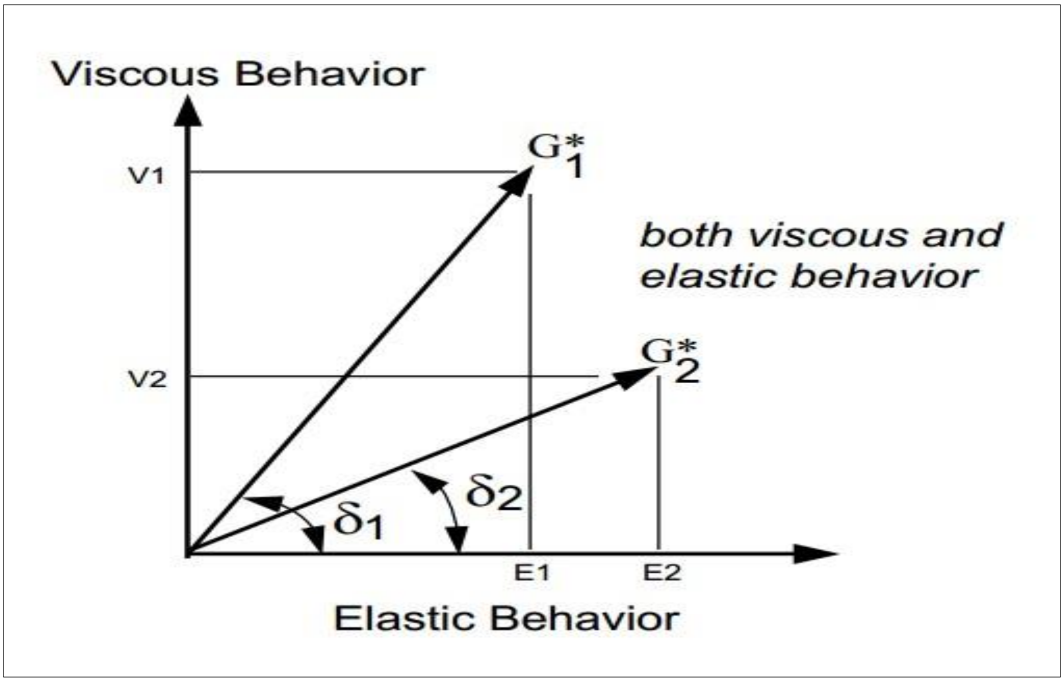



*Figure 2-2. Viscous and Elastic Behavior* ***[18].***

If the same load is applied to both asphalts, then Asphalt-1 will display more nonrecoverable (permanent) deformation than Asphalt-2 since Asphalt-2 has a relatively large elastic component. This example shows that G*, alone, is not enough to describe asphalt behavior. i.e., the δ value is also needed.

## 2.8 The working principle of DSR

The operation principle of the DSR is straightforward. An asphalt sample is sandwiched between an oscillating spindle and the fixed base. As shown in figure 3 and 4 below the oscillating plate (often called a "spindle") starts at point A and moves to point B. From point B the oscillating plate moves back, passing point A on the way to point C. From point C the plate moves back to point A. This movement, from A to B to C and back to A comprises one cycle.

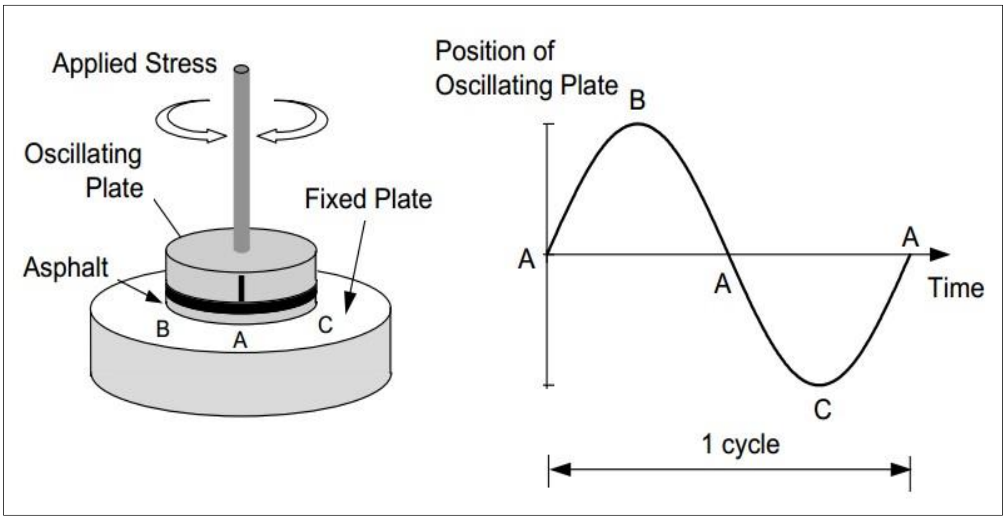



*Figure 2-3. Dynamic Shear Rheometer Geometry* ***[18].***

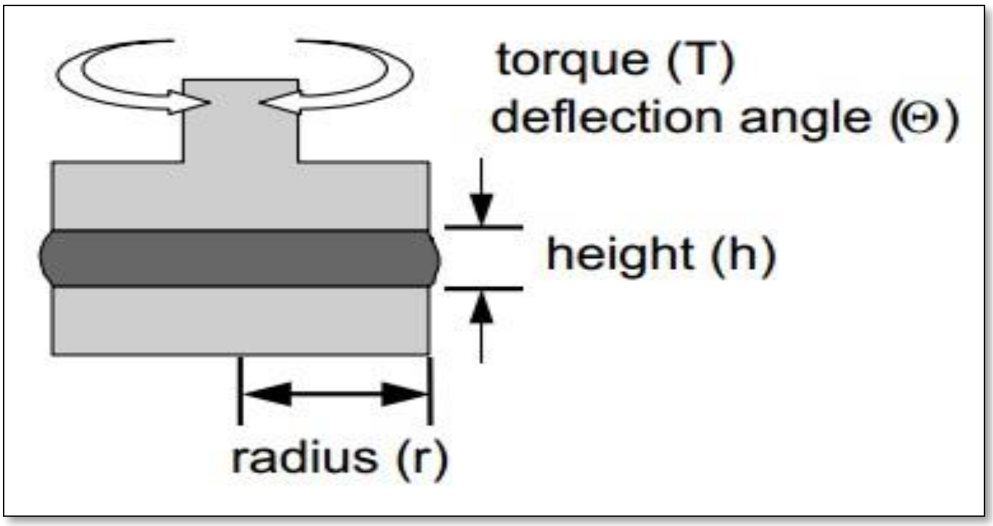


*Figure 2-4. DSR Geometric Parameters* ***[18]***

Based on the geometry and the applied torque, the formulas used by the rheometer software to calculate $\tau_{max}$ and $\Upsilon_{max}$ are:

- ❖ $\tau_{max} = 2T/\pi r^3$
- ❖ $\Upsilon_{max} = \Theta r/h$
- ❖ Where, T = maximum applied torque
- ❖ Θ = deflection (rotation) angle
- ❖ r = radius of specimen or plate (12.5 or 4mm)

As the force (or shear stress, τ) is applied to the asphalt by the spindle, the DSR measures the response (or shear strain, γ) of the asphalt to the applied force. If the asphalt were a perfectly elastic material, the response would coincide immediately with the applied force, and the time lag between the two would be zero.

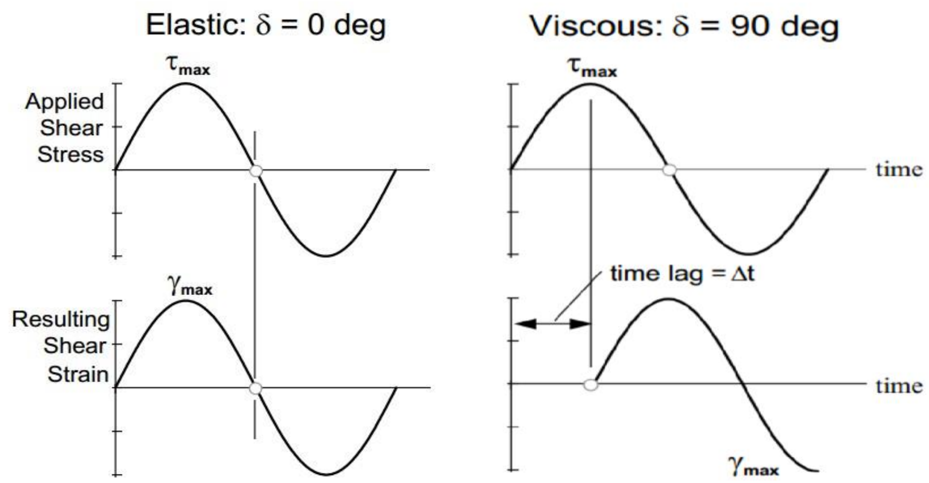



*Figure 2-5. Stress-Strain Output for a constant Stress Rheometer* ***[18].***

The relationship between the applied stress and the resulting strain in the DSR quantifies both types of behavior, and provides information necessary to calculate two important asphalt binder properties: the complex shear modulus (G*) and phase angle (δ)

G* is the ratio of maximum shear stress (τ max) to maximum shear strain (γ max). The time lag between the applied stress and the resulting strain is the phase angle δ. For a perfectly elastic material, the phase angle, δ, is zero, and all of the deformation is temporary. For a viscous material (such as hot asphalt), the phase angle approaches 90 degrees, and all of the deformation is permanent. In the DSR, a visco-elastic material such as asphalt at normal service temperatures displays a stress-strain response between the two extremes, as shown below.

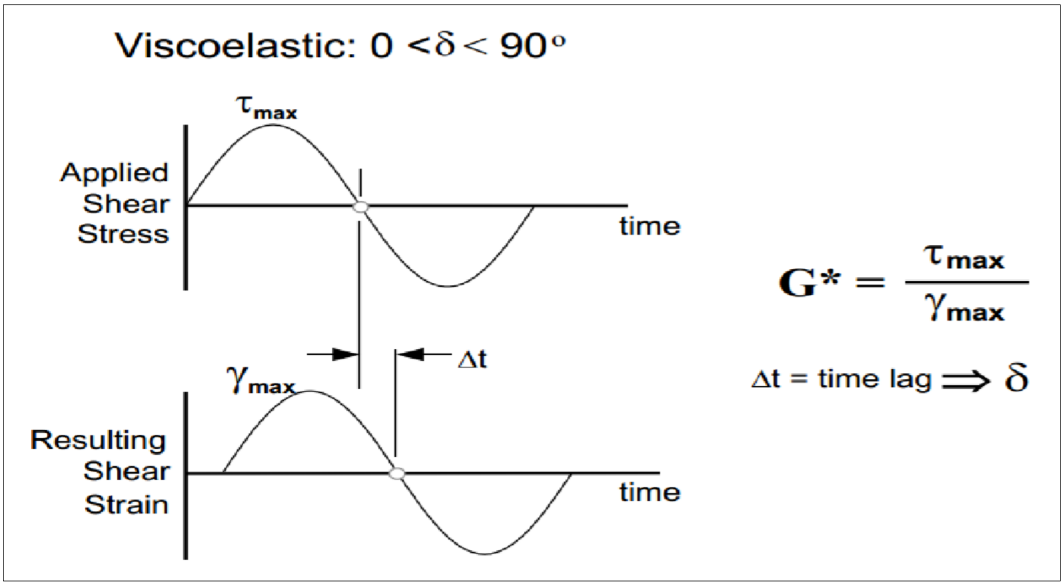



*Figure 2-6. Stress-Strain Response of a Viscoelastic Material* ***[18].***

Conceptually, this kind of response to load can be related to an automobile shock absorbing system. These systems contain a spring and a liquid filled cylinder. The spring is elastic and returns the car to the original position after hitting a bump. The viscous liquid within the cylinder dampens the force of the spring and its reaction to the bump. Any force exerted on the car causes a parallel reaction in both the spring and the cylinder. In hot mix asphalt, the spring represents the immediate elastic response of both the asphalt and the aggregate. The cylinder symbolizes the slower, viscous reaction of the asphalt, particularly in warmer temperatures. Most of the response is elastic or viscoelastic, (recoverable with time), while some of the response is plastic and nonrecoverable.

Now the question is how this property would be connected with rutting or permanent deformation. When we consider a single loading phenomenon with specific stress, loading time and temperature then the resulting deformation will remain partially unrecovered as shown in figure 7 below.

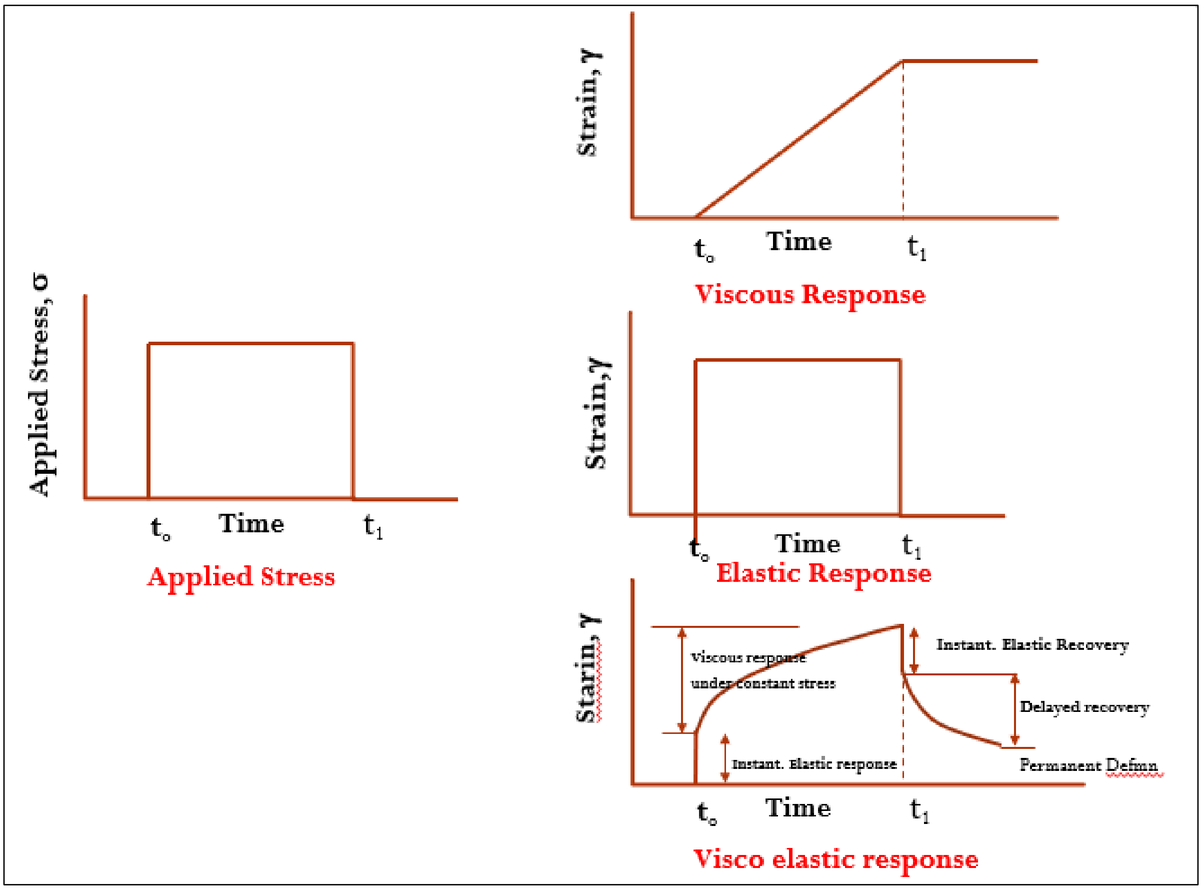



*Figure 2-7. Visco-Elastic Material under Specific Loading Condition [11].*

It is therefore the accumulation of all the unrecovered deformation due to number of loads that will be finally expressed as rutting.

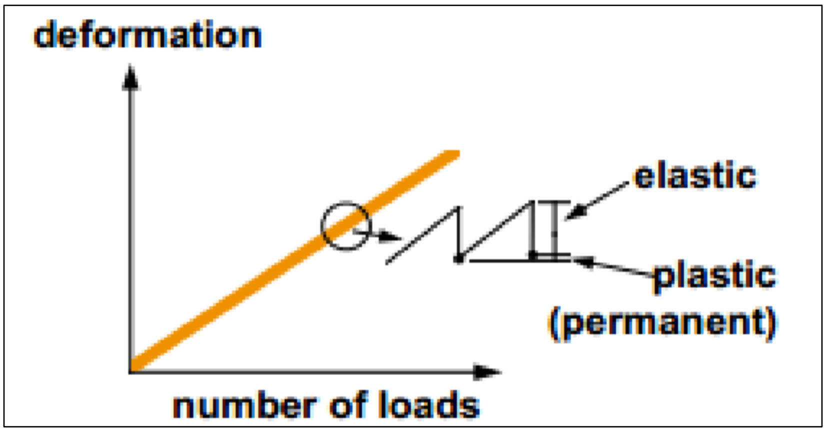



*Figure 2-8. Deformation Due to Number of Loads* ***[6].***

## 2.9 Bitumen Ageing

Aging is the change in structures and composition of asphalt molecules that results hardening and embrittlement of binders during construction and service life of pavements.

There are two mechanisms of aging, irreversible and reversible. The main one, irreversible aging is characterized by chemical changes due to loss of volatiles and oxidation ***[22].*** The reaction of asphalt molecules with oxygen from the environment known as oxidation causes a more brittle structure and that is the origin of the terms "oxidative hardening" or "age hardening". The other irreversible form of hardening which occurs during hot mixing and construction is called "volatilization." At high temperatures, volatile components evaporate from the asphalt. These light, oil-like components, if allowed to remain, would otherwise soften the asphalt.

The reversible phenomenon called "physical hardening" occurs when asphalt cement is exposed to low temperatures for long periods. As the temperature falls, asphalt shrinks in volume and there is an accompanying increase in asphalt hardness. Physical hardening is more pronounced at temperatures less than $0^oC$ and must be considered when testing asphalt cements at very low temperatures ***[23].***

There are two distinct phases of aging, short-term aging during the construction phase of an asphalt pavement and long-term aging during the service life. Short-term aging begins at the mixing plant and ends when the compacted pavement has cooled; long-term aging proceeds thereafter ***[23].***

To simulate or test aging, several methods are employed. For the age hardening occurring during plant mixing and lay-down the most utilized test is Rolling Thin Film Oven Test (RTFOT, AASHTO T 240 & ASTM D-2872). And to simulate long-term ageing during service the Pressure Ageing Test (PAV, AASHTO PP1) was adopted in SHRP binder specifications ***[22, 23].***

One of the roles of aging tests is to evaluate susceptibility of a mixture for aging at its initial condition and the likely performance in aged condition. The second significant role is to enable specimens to be prepared for the accelerated performance tests (fatigue, rutting, and thermal cracking), Moreover it is very important for specification preparation of binders ***[21].***

The risk of rutting due to aging can be evaluated in different ways. According to SHRP specifications ***[2].*** The ratio G*/sinδ at 1.6 Hz presents the Effect of the module with the phase shift value. SHRP specifications set indirectly and by means of the temperature, a minimum value is 1 KPa report before RTFOT and a value of 2.2 KPa after RTFOT. The higher Temperature aging indices (TAI) was proposed to indicate the effect of RTFO aging in rheological properties with respect to un-aged condition at high temperatures ***[15].*** The indices use the measured rutting parameters at high in-service temperatures as ratio of G*/sinδ (RTFO) to G*/sinδ (Org). The index values show that when the temperature of aging increases, the rutting resistance is better ***[18].***

## 2.10 Bitumen Modification

The materials most used in bitumen modification are thermoplastic polymers, rubber and rubber resins, as well as thermoplastic elastomers.

Thermoplastic polymers have the ability to soften with increasing temperature and solidify when the temperature drops. Such phenomena can be explained by their linear molecular structure. Heating weakens the bond between the molecules, turning thermoplastics into a soft viscous material. Among the thermoplastic polymers for bitumen modification are polyethylene (PE), polypropylene (PP), polyvinyl chloride (PVC), polystyrene (PS), viscoelastic and thermoplastics.

Crumb rubber and rubber resin differ from other polymers in their ability to elongate up to 10 times with an applied load and to return to the initial state when the load is removed. This ability is due to the structure of rubber macromolecules: coils which extend like simple elastic strings. Also, rubber molecules are long like ropes that bend and coil randomly.

Rubber modifiers that improve the properties of bitumen are styrene butadiene rubber (SBR) and ethylene-propylene polymers, as well as butyl rubber.

Thermoplastic elastomers (TPE) have elastic properties for adhesive application. TPE is produced in many industrialized countries. Modifiers of this type are available in the form of granules and powders.

Styrene butadiene diblock copolymers (SB), styrene-butadiene-styrene polymers (SBS), styrene-isoprene-styrene polymers (SIS) and styrene-ethylene / butylene-styrene polymers (CE / BS). SBS polymers are used in the modification of road and roofing bitumen, SIS polymers are used as mastics for joints and cracks, and CE / BS polymers increase resistance of bitumen in unfavorable climates.

## 2.11 Benefits of bitumen modification

Bitumen modification offers a construction material with the following performance characteristics:

- ✓ binds well with mineral materials;
- ✓ flexible and elastic at low-temperature, resists deformation at high temperatures;
- ✓ quickly absorbs thermal and mechanical stresses in asphalt layers of the pavement;
- ✓ resistant to fatigue loads due to temperature variations;
- ✓ wide plasticity interval and wider service temperature interval;
- ✓ better mechanical properties and hardness.

Conventional bitumen has a limited range of rheological properties and durability that are not sufficient to resist pavement distresses. Therefore, to minimize the damage of pavement surface and improve durability of flexible pavement, the conventional bitumen needs to be improved in regards with performance related properties, such as resistance to permanent deformation (rutting) and fatigue cracking.

Currently, the most commonly used polymer for bitumen modification is the styrene–butadiene–styrene (SBS) followed by other polymers such as styrene butadiene rubber (SBR), ethylene vinyl acetate (EVA) and polyethylene.

SBS block copolymers are classified as elastomers that increase the elasticity of bitumen and they are probably the most appropriate polymers for bitumen modification. SBS copolymers derive their strength and elasticity from physical and cross linking of the molecules into a three-dimensional network. The polystyrene end blocks impart the strength to the polymer while the polybutadiene rubbery matrix blocks give the material its exceptional viscosity. When SBS is blended with bitumen, the elastomeric phase of the SBS copolymer absorbs the oil fractions from the bitumen and swells up to nine times as much as its initial volume. At suitable SBS concentration, a continuous polymer phase is formed throughout the polymer modified bitumen (PMB) and significantly modifies the base bitumen properties.

EVA based polymers are classified as plastomer that modify bitumen by forming a tough, rigid, three-dimensional network to resist deformation. Their characteristics lie between those of low-density polyethylene, semi-rigid, translucent product and those of a transparent and rubbery material similar to plasticized polyvinyl chloride (PVC) and certain types of rubbers. This type of polymers has revealed as good modifiers which improve permanent deformation and thermal cracking.

Polymers are usually provided in the form of pellets or powder which can be subsequently diluted to the required polymer content by blending with base bitumen by using low to high shear mixer. Blending pellets with base bitumen results in a special polymer concentration suitable for different applications. In spite of the significant research which has been carried out related to the SBS and EVA modified PMBs in road applications, more studies have to be undertaken on the compatibility and in the interaction between the SBS, EVA polymer and the base bitumen ***[28].***

## 2.12 Asphalt Additives

Asphalt modifier is a material, which would normally be added to the binder or the mixtures to improve its properties. Due to increasing traffic load, increasing traffic and changing environmental condition, conventional bitumen fails to satisfy performance requirement. Best alternative is to modify conventional bitumen with different modifiers. Modification offers one solution to overcome the pavement distress deficiencies of bitumen and there by improve the performance of asphalt concrete pavement.

The use of modified bitumen's to achieve better asphalt pavement performance has been observed for a long time ***[26].***

The main objective of the bitumen improvement is to produce ideal modified bitumen's materials with high resistance to permanent deformation, and fatigue cracking.

## 2.13 The Need of asphalt additives

There are many researchers looking for the reasons to modify bituminous materials. The main reasons to modify bituminous materials with different type of additives could be summarized as follows ***[1].***

- ✓ To obtain softer blends at low service temperatures and reduce cracking
- ✓ To reach stiffer blends at high temperatures and reduce rutting
- ✓ To increase the stability and the strength of mixtures
- ✓ To improve fatigue resistance of blends
- ✓ To reduce structural thickness of pavements.

The technical reasons for using modifiers in asphalt concrete mixtures are to produce stiffer mixes at high service temperature to resist rutting as well as to obtain softer mixtures at low temperature to minimize thermal creaking and improve fatigue resistance of asphalt pavement ***[8].***

## 2.14 Historical Background of Anyways Natural Soil Stabilizer (ANSS)

Anyways Solid Environmental Solutions Ltd. is a global leader in providing soil stabilization products to the infrastructure and development sectors. It has been established in 1999.

The company has been providing solutions to road and infrastructure projects since 2000 in almost every continent in the world. In recent years it has focused its activities in Africa but has continued to provide solutions also to projects in Latin America, North America Middle East and Australia. Anyways has at the moment two production centers, one in Israel and the other in South Africa. It has permanent offices in Israel, South Africa, Canada, Ethiopia and representation in Kenya.

The solutions of Anyway are widely used in many road projects in South Africa (haulage roads in mines, developments of new cities and upgrading of urban roads), it is also being implemented in many projects in Israel, from North to South, in projects in Ethiopia, Kenya, Angola, Burkina Faso, Ivory Coast, Eritrea, Mozambique, Namibia, Nigeria and in Europe in Italy, Spain and Portugal. It has also been implemented in projects in Latin America, in Brazil, Ecuador and Peru.

The use of Anyway Natural Soil Stabilizer (ANSS) in road projects is recognized as an extremely cost-effective method of converting poor quality soil into a strong impermeable layer. It permits the construction of pavement layers, embankments and reinforced earth structures in areas where they were not previously economically viable, while saving significant sums of money.

ANSS is a calcium driven, inorganic soil stabilizer patented worldwide. Its specific formulation allows for stabilization of a broad range of materials without compromising the quality of the result ***[17].***

## *2.15* Uses of ANSS in road Construction

The main components that are used to formulate ANSS are a series of inorganic hydration activated powders. It is composed of a specific type of cement, a lime, several pozzolans, rate governing additives, and a unique polypropylene fiber other name poly (propene) (used as concert additives to increase strength and reduce cracking). The specific

formulation allows for the individuality of the components to contribute to the reaction process, but also act holistically contributing of the stabilization process.
The theory behind their reactivity is quite simple, but the chemistry of each individual powder differs and the collaborative reaction is quite complex. Each component reacts individually while also contributing to the broader stabilization reaction. Each component contained in the stabilizer has its own series of reactions that occur at varying rates, which can be broken down into initial, short term and long-term reactions ***[17].***

## 2.16 Effect of Lime on asphalt

Lime has been used in hot mix asphalt (HMA) to reduce moisture sensitivity and stripping since 1910 in the United States. While hydrated lime has long been an acknowledged anti-strip additive for asphalt pavements, recent studies confirm that lime imparts other important benefits:

- ✓ It stiffens the binder and HMA to resist rutting;
- ✓ It improves toughness and resistance to fracture growth at low temperatures;
- ✓ Lime changes oxidation chemistry in the binder to reduce age hardening; and
- ✓ Lime alters clay fines to improve moisture stability and durability.

Lime is also useful to upgrade marginal aggregates. In addition to the chemical effects that lime imparts to reduce stripping potential and the aging Effect resulting from oxidative hardening, the "filler effect" of lime improves resistance to high-temperature rutting and adds fracture toughness at low temperatures. ***[11].***

## 2.17 Rheological Data Presentation

Data obtained from DSR (rheological data) can be represented in different forms to analyze the rheological properties of a binder in different ways.

### 1. Isochronal Plot

Isochronal Plot is a curve representing the behavior of visco-elastic function at a constant frequency. Curves of complex modulus or phase angle versus temperature at constant frequency are isochrones ***[9, 20].*** Isochronal plot helps to compare complex modulus or phase angle at different temperatures and also to evaluate other properties like temperature susceptibility ***[9]***.

### 2. Isothermal Plot

Isothermal plot can be defined as a curve or an equation representing the behavior of visco-elastic function at a constant temperature. Curves of complex modulus or phase angle versus frequency at a constant temperature are isotherms ***[4].*** The plot helps to compare different visco-elastic properties mainly G* & δ at unvarying temperature but at a range of frequency. i.e., it uses to study time dependency of a material ***[9].***

### 3. Black space Diagram

This can be described as a graph of log complex modulus ***[20]*** plotted as a function of phase angles. The diagram is useful to plot the two important rheological parameters (G* & δ) in a single curve without referring frequencies and temperatures ***[25].*** The decrease in complex shear modulus (G*) with the increase in phase angle δ depends on the binder types. This implies that black diagram depicts whether the binder is modified or conventional ***[25].*** Also, it is important to evaluate the quality of test data ***[20.25].***

### 4. Master Curves

Master curves are constructed using the principle of time temperature superposition because of the relationship between temperatures and frequencies (times of loading ***[9].***

From the data collected over a range of temperatures and frequencies we can have several rheological graphs. To represent those graphs with one master curve a standard reference temperature must be selected. Then, the data at all other temperatures are shifted relative to this reference temperature and at a reduced frequency until a smooth curve is generated.

The master curves of the complex modulus, storage modulus, loss modulus and phase angle with the change in frequency can be constructed in this manner ***[2].*** And this master curve is useful to obtain interpolated values of property of any combination of temperature (T) or frequency inside the range covered by the measurement. Master curves allow the rheological data to be presented over a wide range of frequencies and temperatures in one plot. Therefore, to avoid presenting a large number of graphs, the results are mainly presented and analyzed as master curves ***[9].***

## 2.18 Summary

Generally, the literature review describes overall performance of asphalt concrete pavement depends on the bitumen properties, asphalt concrete mixtures volumetric properties and external factors. And how modification or replacement of asphalt binder was introduced and the main reasons behind it. Basically, this research mainly focuses on addition of ANSS in HMA as an alternative binder. and it will be beneficial to apply it in our country. After reviewing the behavior of bitumen related to addition of ANSS in the asphalt mix materials are prepared for conducting laboratory tests.

# CHAPTER 3 METHODOLOGY

## 3.1 Introduction

This chapter provides information on the research method and procedure of this thesis. The research methodology applied for this study is experimental. The experimental method describes the types of materials to be tested, sample preparation, types of tests, test procedures and related things under this chapter. Both applied and basic research is also used.

## 3.2 Materials

The materials prepared for different experimental works are five in type. These are unmodified 60/70 penetration grade bitumen which is the base or control material and bitumen modified with 3%, 6 %, 9% and 12 % ANSS.

## 3.3 Experimental Plan

These tests were performed on asphalt binder mixed with different percentages of ANSS varying three percent gap by weight of the bitumen. According to AASHTO T-316 or ASTM D 4402 First the asphalt binder was heated to a temperature of 135-170 $^{o}$C then the necessary amount of ANSS was added to asphalt binder by contentiously stirring the mixture for 15-20 minutes at a constant temperature to ensure good homogeneity, then the following different tests were performed.

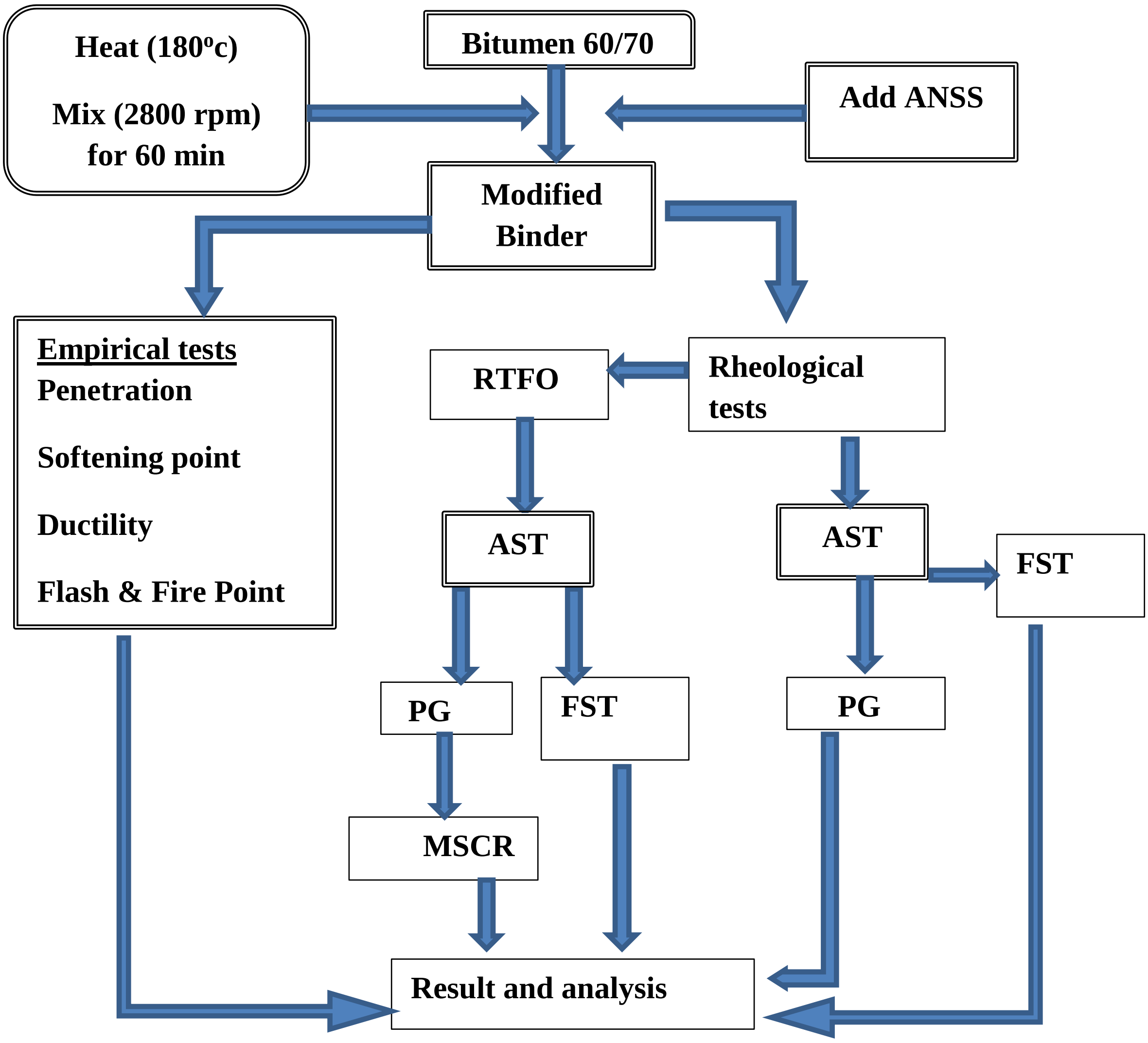



*Figure 3-1. Experimental Flow Chart*

## 3.4 Conventional Tests

### 3.4.1 Penetration Test

Penetration measures the consistency (hardness or softness) of asphalt binder. The standard test method for penetration of bituminous materials is described under ASTM-D5 or AASHTO T 49. The penetration is determined by measuring the distance in tenths of millimeter that a standard needle vertically penetrates a sample of the material under a specified load at a specified temperature within a specific period of time. In normal condition 100g of loading at a temperature of 25°C and 5sec. of testing time is common. Higher values of penetration indicate softer consistency and the lower the value the harder the bitumen is. Penetration can also be used as indirect measure of viscosity by correlating using empirical formulation.

*Figure 3-2. Experimental setup of Penetration Test*

### 3.4.2 Softening Point

Standard method of test for softening point (Ring and Ball Apparatus) is stated in ASTM D 36 or AASHTO T 53. To carry out the test two disks of specimens are prepared using shouldered brass rings. Then the rings (samples) loaded with a 3.5g steel ball at the center of each ring (sample) will be placed in an assembly and immersed in a beaker of water. The water initially at 5$^o$c will be heated at a controlled rate usually 5$^o$c per minute. Finally, the softening point will be the mean of temperatures recorded when the two balls enveloped with soft bitumen touches the lower plate 25mm below the rings.

Softening point helps to classify bitumen, check uniformity and signify its tendency to flow at elevated temperature. Higher softening point indicates the lower temperature susceptibility and preferred in warm climates.

*Figure 3-3. Experimental setup of Softening Point Test*

### 3.4.3 Ductility

According to ASTM D113 or AASHTO T 51, which describes the standard test method, the ductility of bituminous materials is measured by a distance in centimeters to which the standard briquette of bituminous sample can be stretched before the thread breaks when pulled apart at a specified speed and a specified temperature. If no special reason, the standard test temperature which will be maintained using water bath is 25°C (77°F) and the rate of pull to elongate the sample is 5cm per minute. The ductility test measures the adhesive property of bitumen and its ability to stretch.

*Figure 3-4. Experimental setup of Ductility Test*

### 3.4.4 Fire and flash point-

According to (ASTM D92) which Determines the temperature at which asphalt materials safely be heated. Heating asphalt above the softening point to able it to fill the test cup and Fix the thermometer inside the sample. (don't touch the cup bottom) then Start the test heater to heat in a rate of (5-6) $^{0}$C/min. Before the expected Flash point by about (28$^{0}$C) start to close the flame from the samples surface each (1$^{0}$C) until (104$^{0}$C) then after (104$^{0}$C) but in intervals of each (3$^{0}$C) Compute the flash and fire points when they happened. Flash point is that point of temperature at which the asphalt will flash for one second under specific situation. Fire point is minimum point of temperature at which the asphalt will fire (burn) for five seconds under specific situation.

*Figure 3-5. Experimental Setup of Flash and Fire Point Test*

## 3.5 Dynamic Shear Rheometer (DSR) Tests

Several fundamental and research-based binder tests can be carried out using dynamic shear rheometer. From those different tests Amplitude sweep test, Frequency sweep test, performance grade determinations and the multiple stress creep recovery have been performed.

### 3.5.1 Sample Preparation for Rheological Test

For the purpose of this study is to test rheological behaviors of the mix at different frequency and temperature. First the asphalt binder is heated until it is sufficiently fluid to pour and to prepare the test specimens as shown in the picture below to make suitable for both 25mm of high temperature tests and for intermediate temperature 8mm plate the specimens were prepared as shown in figure below.

*Figure 3-6. Specimen Prepared for DSR Test*

### 3.5.2 Basic Test Procedure

The Standard test method for determining the rheological properties of asphalt binder using dynamic shear rheometer is described in AASHTO T315-10.

First the asphalt binder is heated until it is sufficiently fluid to pour and to prepare the test specimens. Then a small sample of asphalt binder is sandwiched between two plates. But before placing the sample the DSR is set to a particular temperature; this preheats the upper and lower plates, which allows the specimen to adhere to them. Depending upon the type of asphalt binder being tested the test temperature, specimen size and plate diameter varies. The DSR apparatus used is as shown in figure 15.

*Figure 3-7. Dynamic Shear Rheometer Setup*

For a sample 0.04 inches (1 mm) thick and 1 inch (25 mm) in diameter, test temperatures greater than 115°F (46°C) are used whereas for a sample 0.08 inches (2 mm) thick and 0.315 inches (8 mm) in diameter, test temperatures between 39°F and 104°F (4°C and 40°C) are used.

To suit the desired size of specimen the upper spindle is lowered until the gap between the plates equals the test gap plus 0.002 inches (0.05 mm). Due to the compression, excess material will come out which is then trimmed around the edge of the test plates using a heated trimming tool. The test plates further moved together to the selected testing gap by eliminating the additional 0.05 gap. This creates a slight bulge in the asphalt binder specimen's perimeter.

The test specimen is kept at near constant temperature by heating and cooling a surrounding environmental chamber. The test is started up only after the specimen has been at the desired temperature for at least 10 minutes. The instrument measures the maximum applied stress, the resulting maximum strain, and the time lag between them while the top plate oscillates in a sinusoidal waveform. The calculation of the complex modulus (G*) and phase angle (δ) is done automatically with the help of the software. Based on the material being tested (e.g, unaged binder, RTFO residue or PAV residue) the determination of a target torque at which to rotate the upper plate is carried out using the DSR software. To ensure that the measurements are within the specimen's region of linear behavior this torque is chosen. The range of the phase angle (δ), from about 50 to 90°, and while that of the complex modulus (G*), from about 0.07 to 0.87 psi (500 to 6000 Pa), are the typical values obtained from the DSR for asphalt binders. The complete viscous behavior is essentially the δ of 90°. The ANSS -modified asphalt binders usually exhibit a higher G* and a lower δ value. Hence, it is meant that compared to the unmodified asphalt cements they turned out to be more elastic and a bit stiffer.

### 3.5.3 Tests on the DSR

#### *3.5.3.1 Amplitude Sweep Test*

Amplitude sweep is an oscillatory DSR test with variable stress or strain amplitude at constant frequency. The main or the sole purpose of this test is to determine the Linear Visco Elastic Range (LVR) of a visco-elastic material. The linear visco elastic part is the region where the applied oscillation is nondestructive. In most cases log-log graph on the same scale is plotted as strain in the x-axis and shear modulus in the y-axis. The complex shear modulus G* versus strain plot was used to determine the linear visco-elastic (LVE) region.

Several options are there to determine the limit of the LVE range.

1. Automatic analysis using a software analysis program: After the user has defined the bandwidth of tolerance, a special analysis program determines the limiting value.
2. Visual or manual analysis:

   a) By simply observing the curve the limiting strain value ($\gamma_L$) can be taken at the point where the curve noticeably falls. To make this easier a straight line (analysis tangent) can be drawn along the level of the plateau value.

   b) From the data table considering the G* which does not deviate significantly from the plateau value in the LVE range, the corresponding strain can be taken as limiting value. During this process the bandwidth of the tolerated deviation has to be defined by the user as 1%, 5%, or 10% in most cases by considering the type of binder.

This study has followed this manual analysis method and 5% was chosen as deviation tolerance. In this way all those values which are below 95% of the plateau value are considered to be outside the LVE range.
The amplitude sweep test was carried out following the test standard AASHTO T 315 at a constant frequency of 10rad/sec at specific test temperatures ($10^0$c,21.1 $^o$c, 37.8 $^o$c & 54.4$^o$c). The test was in shear stress control mode with minimum shear stress 100pa and maximum shear stress 90000pa.

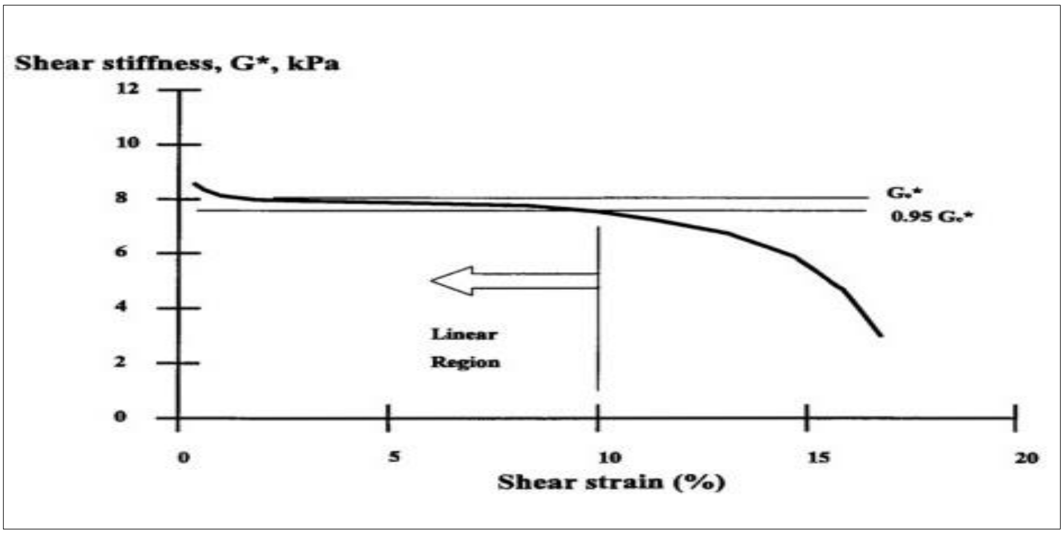



*Figure 3-8. Amplitude Sweep to Determine Visco-Elastic Region* ***[7]***

#### *3.5.3.2 Frequency Sweep Test*

Frequency sweep is also an oscillatory test with variable frequency and constant amplitude values. Using this test time dependent shear behavior can be examined. Short-term behavior is simulated by rapid movements (at high frequencies) and long-term behaviors by slow movements (at low frequencies). Frequency sweep helps to evaluate the rheological property of visco-elastic material by developing a master curve from isothermal plots of the test result.

The frequency sweep test was conducted in a strain-controlled mode varying from 0.1Hz to 25 Hz. From the amplitude sweep test the limiting strain value was found to be 2%. Based on this result the strain input value taken for frequency sweep test was 1%, which was chosen to examine the binder well within linear visco-elastic range.

#### *3.5.3.3 Multiple Stress Creep Recovery (MSCR) Test*

The Multiple Stress Creep Recovery (MSCR) test is the latest improvement to the Superpave Performance Graded (PG) Asphalt Binder specification. This new test and specification listed as AASHTO T350 and AASHTO M332 provide the user with a new high temperature binder specification that more accurately indicates the rutting performance of the asphalt binder and is blind to binder modification. A major benefit of the new MSCR test is that it eliminates the need to run tests such as elastic recovery and phase angle procedures designed specifically to indicate polymer modification of asphalt binders. Several studies have shown that the G*/sin(δ) based specification does not correlate well with field performance ***[16].***

The test protocol (AASHTO T350) requires that a 25-mm diameter and 1-mm thick asphalt specimen is subjected to 10 cycles of one second creep loading followed by 9 seconds rest period at stress levels of 100 Pa and 3200Pa at the high PG temperature using a DSR. In this way 20 cycles at the 0.1-kPa stress level followed by 10 cycles at the 3.2-kPa stress level for a total of 30 cycles will be done. The first 10 cycles at 0.1 kPa will be used for conditioning the specimen. There are no rest periods between creep and recovery cycles or changes in stress level. The total time required for completing the two-step creep and recovery test is 300s. the sample has to be residue from T 240 (Rolling Thin-Film Oven Test). From the test we can determine the following main parameters,

i. Non-recoverable creep compliance
ii. Percent difference between non-recoverable creep compliance
iii. Average percent recovery Percent difference in recovery
iv. MSCR based new PG grade and test temperature

## 3.6 Performance Grade Determination

The Superpave is a binder specification and mix design procedure developed by Strategic Highway Research Program (SHRP). This binder specification system works based on climate at which the pavement is expected to serve by evaluating the contribution of the binder in resistance to permanent deformation, low temperature cracking and fatigue cracking in asphalt pavements.

According to the superpave, to carry out performance grade determination new set of tests of physical properties at a range of temperatures must be carried out. The performance grade (PG) of the binder is designated as PGxx-yy, where xx represents the average seven days maximum temperature and yy represents the minimum temperature. In this grading system, even though tests are to be conducted at different temperatures, requirements of Physical properties will remain the same.

*Table 1. Set of Binder Test According to Superpave*

| **Description** | **Test Type** | **Aging Condition** | **Test Temperature** |
|---|---|---|---|
| Construction | Rotational Viscometer | No aging | Very high |
| Rutting | DSR (25mm plate) | No aging | High |
| | | RTFO aged | |
| Fatigue cracking | DSR (8mm plate) | PAV aged (after RTFO) | Intermediate |
| Thermal cracking | DTT & BBR | PAV aged (after RTFO) | Low |

In view of the above and based on the objective of the study, the PG test is carried out at high temperature considering rutting only. Thus the types of samples were original binder and RTFO aged binder. And the test plates used were 25mm in diameter for a 1mm thickness of specimen. There were two basic reasons for why only high temperature tests. The first is because the objective of the study focuses on rutting. The second reason was the unavailability of pressure aging vessel (PAV) to carry out long term ageing for intermediate and low temperature tests.

## 3.7 Test Temperature and Work Plan

To plan the work, it is necessary to know the number of test repetitions considering the applicable standard test temperatures for this study.

### 3.7.1 Test temperature

Unlike conventional tests which are single temperature, according to superpave fundamental binder tests must be carried out at different temperatures. There are different standard test temperatures, which require determining and selecting the relevant and specific test temperatures considering the climate and the test type. Here we can consider two standards of test temperatures. One is as per the superpave binder specification, high test temperatures are 46 $^{o}$c, 52 $^{o}$c, 58 $^{o}$c, 64 $^{o}$c, 70 $^{o}$c, 76 $^{o}$c and 82 $^{o}$c. There are also intermediate and low-test temperatures. The second is the MEPDG standard test temperatures which mostly are used to conduct Amplitude Sweep and Frequency Sweep tests. The common temperatures in this case are 54.4$^o$c (130$^o$F), 37.8$^o$c (100$^o$F), 21.1$^o$c (70$^o$F), 4.4 $^o$c (40$^o$F) and 10 $^o$c (-10 $^o$F).

### 3.7.2 Work Plan

The work plan is organized taking minimum test replicates and minimum spectrum of test temperatures to minimize the lab work as much as possible.

*Table 2. Experimental Work Plan*

| SN | Description | | | Number of Tests | | |
|---|---|---|---|---|---|---|
| | **Sample Type** | **Sample Codition** | **Activity/Test** | **Test Replicate** | **# Test temperatures** | **# Tests** |
| **1** | 0,3,6,9,12 % | Unaged | **Conventional tests** | | | |
| | | | Pentration | 4 | 1 | 4 |
| | | | Ductility | 4 | 1 | 4 |
| | | | Softing Point | 4 | 1 | 4 |
| | | | Fire and Flash Point | 4 | 1 | 4 |
| | | | **Rheological Test** | | | |
| **2** | 0% | Unaged | PG determination | 2 | 3 | 6 |
| | | | AST (@ 21.1, 37.8 & 54.4oc) | 2 | 3 | 6 |
| | | | FST (@ 21.1, 37.8 & 54.4 oc) | 2 | 3 | 6 |
| | | | RTFOT | | | |
| | | RTFO Aged | AST (@ 21.1, 37.8 & 54.4oc) | 2 | 3 | 6 |
| | | | FST (@ 21.1, 37.8 & 54.4 oc) | 2 | 3 | 6 |
| | | | PG determination | 3 | 3 | 9 |
| | | | MSCR (@ 21.1, 37.8 & 54.4oc) | 2 | 3 | 6 |
| | | | MSCR (@ 52, 58 & 64oc) | 2 | 3 | 6 |
| **3** | 3% | | Blending | | | |
| | | Unaged | AST (@ 21.1, 37.8 & 54.4oc) | 2 | 3 | 6 |
| | | | FST (@ 21.1, 37.8 & 54.4 oc) | 2 | 3 | 6 |
| | | | PG determination | 2 | 3 | 6 |
| | | | RTFOT | | | |
| | | RTFO Aged | AST (@ 21.1, 37.8 & 54.4oc) | 2 | 3 | 6 |
| | | | FST (@ 21.1, 37.8 & 54.4 oc) | 2 | 3 | 6 |

| | | | | | | |
|---|---|---|---|---|---|---|
| | | | PG determination | 2 | 3 | 6 |
| | | | MSCR (@ 21.1, 37.8 & 54.4oc) | 2 | 3 | 6 |
| | | | MSCR (high temp.) | 2 | 3 | 6 |
| **3** | 6% | | Blending | | | |
| | | Unaged | AST (@ 21.1, 37.8 & 54.4oc) | 2 | 3 | 6 |
| | | | FST (@ 21.1, 37.8 & 54.4 oc) | 2 | 3 | 6 |
| | | | PG determination | 2 | 3 | 6 |
| | | | RTFOT | 2 | 3 | 6 |
| | | RTFO Aged | AST (@ 21.1, 37.8 & 54.4oc) | 2 | 3 | 6 |
| | | | FST (@ 21.1, 37.8 & 54.4 oc) | 2 | 3 | 6 |
| | | | PG determination | 2 | 3 | 6 |
| | | | MSCR (@ 21.1, 37.8 & 54.4oc) | 2 | 3 | 6 |
| | | | MSCR (high temp.) | 2 | 3 | 6 |
| **4** | 9% | | Blending | | | |
| | | Unaged | AST (@ 21.1, 37.8 & 54.4oc) | 2 | 3 | 6 |
| | | | FST (@ 21.1, 37.8 & 54.4 oc) | 2 | 3 | 6 |
| | | | PG determination | 2 | 3 | 6 |
| | | | RTFOT | | | |
| | | RTFO Aged | AST (@ 21.1, 37.8 & 54.4oc) | 2 | 3 | 6 |
| | | | FST (@ 21.1, 37.8 & 54.4 oc) | 2 | 3 | 6 |
| | | | PG determination (64,70,76,80) | 2 | 3 | 6 |
| | | | MSCR (@ 21.1, 37.8 & 54.4oc) | 2 | 3 | 6 |
| | | | MSCR (high temp.) | 2 | 3 | 6 |
| **5** | 12% | | Blending | | | |
| | | Unaged | AST (@ 21.1, 37.8 & 54.4oc) | 2 | 3 | 6 |
| | | | FST (@ 21.1, 37.8 & 54.4 oc) | 2 | 3 | 6 |
| | | | PG determination | 2 | 3 | 6 |
| | | | RTFOT | | | |
| | | RTFO Aged | AST (@ 21.1, 37.8 & 54.4oc) | 2 | 3 | 6 |
| | | | FST (@ 21.1, 37.8 & 54.4 oc) | 2 | 3 | 6 |

| | | | | | | |
|---|---|---|---|---|---|---|
| | | | PG determination (64,70,76,80) | 2 | 3 | 6 |
| | | | MSCR (@ 21.1, 37.8 & 54.4oc) | 2 | 3 | 6 |
| | | | MSCR (high temp.) | 2 | 3 | 6 |
| | | | | | **Total # of tests** | **265** |

The test repetition indicted in table 2 above represents the required number of tests for analysis. But what is actually carried out is by far more than that. There were a number of tests carried out to practice the test procedures and the equipment. Also repetitions of tests for inconvenient result and other reasons due to power interruption and personal mistakes were not included below in the table.

# CHAPTER 4 RESULT AND ANALYSIS

## 4.1 Introduction

This chapter presents the result on the asphalt binder property change up on addition of ANSS as a replacer for both binder and mix properties. Then the tests results are used to draw conclusions on the performance of the two types of materials and their ability to meet specifications.

## 4.2 Effect of ANSS on Asphalt Binder

### 4.2.1 Effect of ANSS on Penetration

Figure 4-1-illustrates the penetration test for the base asphalt and ANSS-asphalt. It can be observed that the addition of different percentage of ANSS to the base asphalt reduces its penetration steadily. A reduction in penetration depth as the percent of ANSS increases up to 9 %. but, as the percentage increases to 12 the penetration value slightly increase from 64.9 mm to 65.2 mm. It shows that content of ANSS has a significant effect on penetration value by increasing stiffness of ANSS modified binder. Thus, would make the binder less temperature susceptible and lead to high resistance to permanent deformation like rutting as mentioned by (Liu et al.,2009).

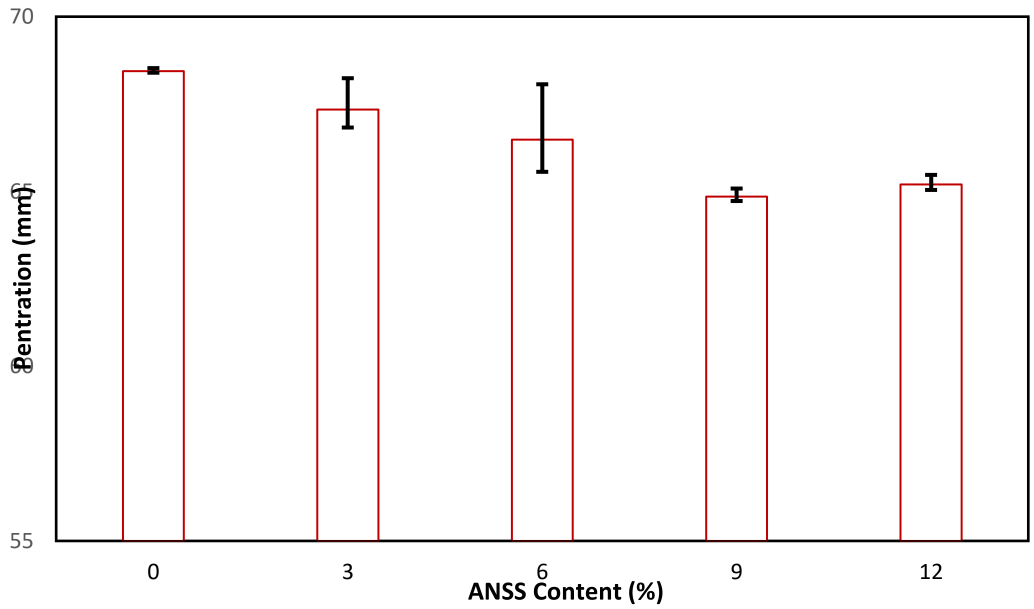



*Figure 4-1. Penetration Test Result*

### 4.2.2 Effect of ANSS on Softening point

Figure 4-2 shows that the softening point property of modified asphalt is increase with increase in percentage of ANSS and slightly decrease as the percentage increase to 12 which implies that addition of ANSS increase resistance to temperature susceptibility as mentioned before in literature review higher softening point is required for better resistance of asphalt pavement temperature. And according to (Diyala et al., 2016) higher softening point of asphalt cement makes better resistance deformation at high temperature as well as protect the mixtures from bleeding.

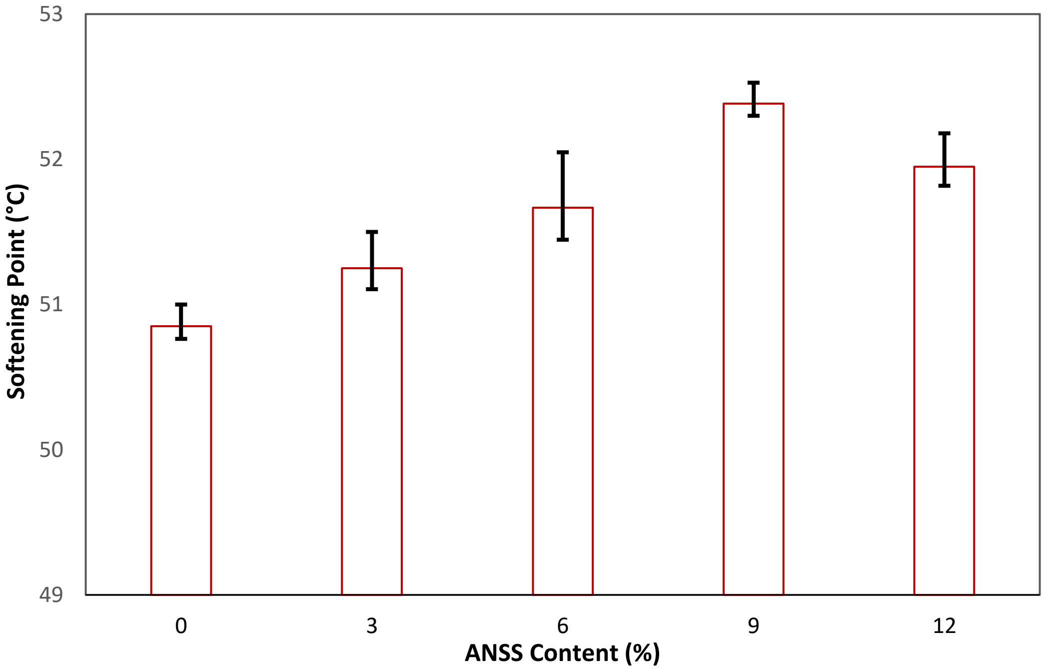



*Figure 4-2. Effect of ANSS on Softening Point*

### 4.2.3 Effect of ANSS on Ductility

The addition ANSS shows no breaking of the mix as the percentage of ANSS increase. From this we can anticipate that the stretching ability of the mix doesn't affect as percentage of ANS increase.

### 4.2.4 Effect of ANSS on Fire and Flash Point

Figure 4-3 shows that an increase in the flash and fire point of bitumen from 305 $^0$c to 307$^0$c was obtained when 3 % ANSS by weight of bitumen is added .similarly, increase was observed the ANSS content of bitumen was increase from 6 % to 12 % with the higher value obtained at 9 % corresponding to 320.5 $^o$c .the increase in flash and fire point attribute to the difference in the ignition temperature of ANSS binder mix .i.e. the

ANSS mix ignite at high temperature .this signifies that the inclusion of ANSS content into bituminous mix will likely reduced associated fire hazard.

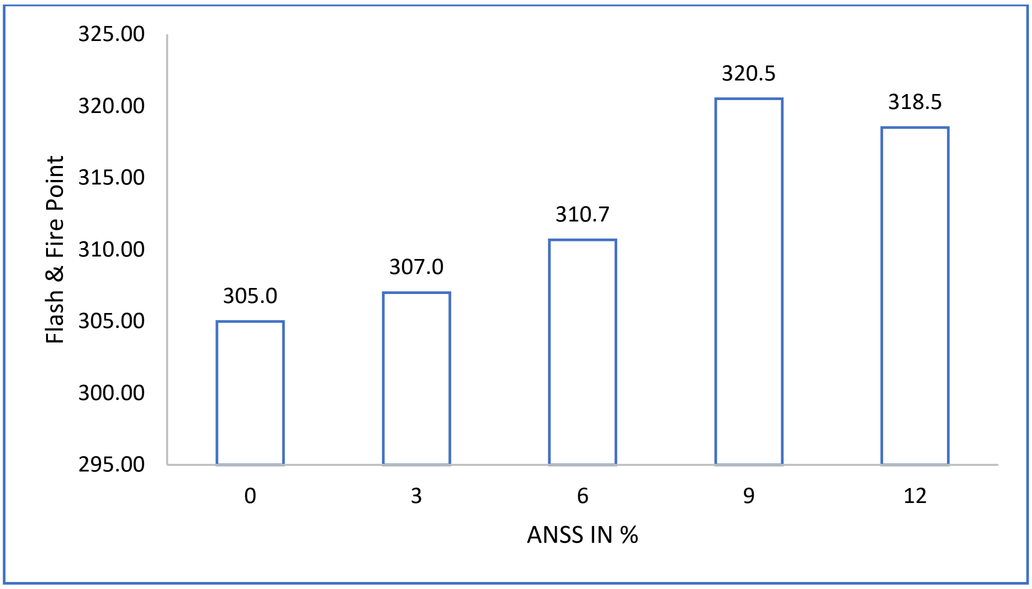



*Figure 4-3. Effect of ANSS on Softening Point*

In summery based on the result of this study addition of ANSS to bitumen binder enhance the physical properties of ANSS modified bitumen binder as indicated by the reduction in penetration and increasing the softening and flash and fire point with no change in ductility. Thus, enhancing ANSS modified bitumen and increase its ability to resist rutting deformation up to 9 % ANSS concentration might be used as modifier of bitumen.

### 4.2.5 The Effect of ANSS on Amplitude Sweep Test

The amplitude sweep tests were carried out for each specimen with similar loading condition ranging from 100 Pa to 90000 Pa in a stress-controlled mode at a constant frequency of 10 rad/sec.

From the AST result, the complex modulus decreased with increasing temperature. This is because as temperature increases the materials will have a larger viscous component there by decreasing the complex modulus and increasing the phase angle. Figure 4-4 shows a typical complex Modulus VS strain for aged 6% ANSS. More table and figures related to AST Effects are presented in **Appendix C.**

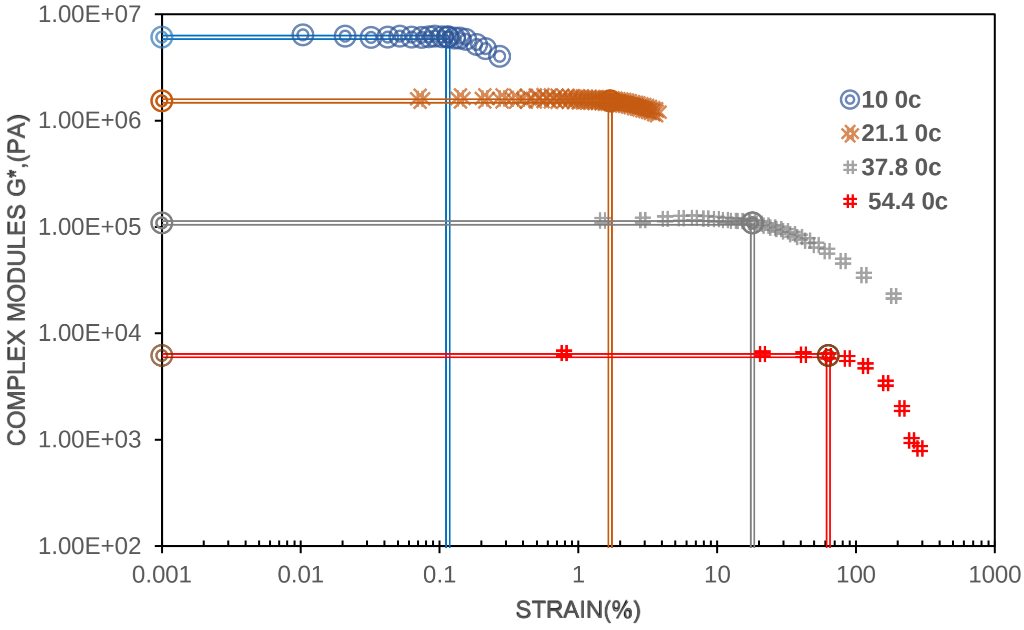



*Figure 4-4. Linear Visco Elastic Range for 6 % ANSS before RTFO*

At 10 °C and 21.1°C the limiting strain value for each sample is minimum and the Effect of the modifier is not significant. furthermore, ANSS mix exhibit dominant elastic behavior with minor differences in stiffness between the binders, all the ANSS mix showed the LVE was independent of complex modulus at low temperatures.

At 37.8°C and 54.4°C temperature higher limiting strain values are observed as the material gets less stiff implies the complex modulus decreased with increasing temperature. Showing ANSS has Effect on the rheological property asphalt binder. the LVE strain limits decreased while the LVE stress limits increased with increasing complex modulus which means viscous behavior dominates binder performance

It is possible to say there is considerable change in limiting strain values after RTFO ageing specially at 54.4 °C. The effect of the modifier is pronounced at higher temperature. More figures and table related to AST Effects are presented in **Appendix B.**

Based on test result represented graphically for each and every curve of amplitude sweep test a horizontal line along 0.95G* will be constructed to intersect the curve at a point. Then the corresponding strain value of that intersection point will be considered to be the limiting stain value (γL). To summarize, it is observed that the amplitude sweep test enables to recognize the responses (change) in stiffness and LVE range due to the three factors temperature, content of modifier and aging.

Most of the time several researchers conduct tests by taking strain value from 1% - 2% and according to this study the maximum limiting strain is found to be 2%, but to be well below the maximum strain or to ensure that the strain taken is unquestionably within the linear visco-elastic region, it is better to take 1% strain for the purpose of this study. If we take 2% as an input for further tests which is near to the nonlinear visco elastic region then we may not be able to get consistent test effects. Therefore 1% strain will be used to carry out frequency sweep tests of this study.

## 4.3 Black Space Diagram

Figure 4-5 illustrate that black diagram which is simply a plot of complex modulus versus phase angle obtained from a dynamic test Therefore, viscoelastic data is plotted over wide range of temperatures and frequencies using the black diagram. Measurement errors, changes in composition, or variations in bitumen structure can cause deviations within the black diagram [30, 31]. This plot is therefore very useful for presenting the effect of ageing or modification of bitumen.

The black space diagrams are helpful to evaluate the quality of the test data. In view of this the graphical data representation for all binders of this study has a good quality since the data of each graph was not dispersed.

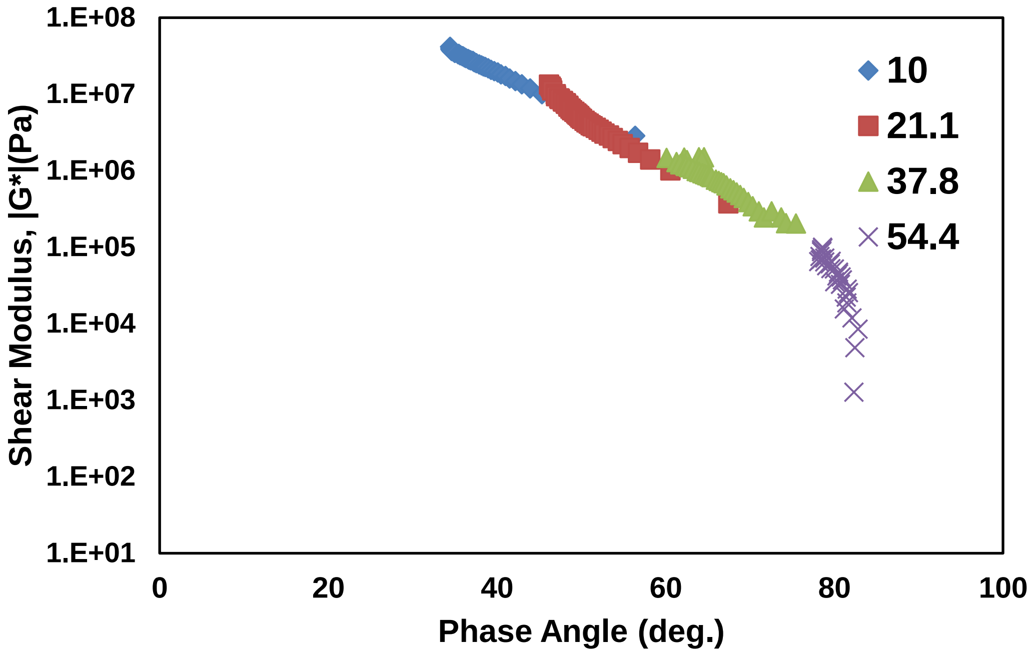



*Figure 4-5. Black Space Diagram for Binder Mixes*

## 4.4 The Effect of ANSS on Frequency Sweep Test

The frequency sweep test was conducted in strain-controlled mode using 1% strain as determined from the AST. The sweep or variation in frequency was set from high to low (25Hz-0.1Hz) in an increasing damaging Effect. Frequency sweep test Effects at $10^0$c, $21.1^o$c, 37.8 $^o$C, and 54.4 $^o$C for all samples both aged and un-aged were determined and organized as stated below.

Figures 4-6 illustrates that, since complex modulus is function of temperature and frequency value of stiffness have increased with the increase in frequency, while it decreased with the increase of the temperature. And the phase angle values have increased with the decrease in frequency, while it increased with the increase of the temperature as it did for AST.

In general, the increase in complex modulus due to the increase of the frequency is based on the fact that the material is in the plastic region at low frequencies (high values for phase angle). More figures related to FST Effects are presented in **Appendix C.**

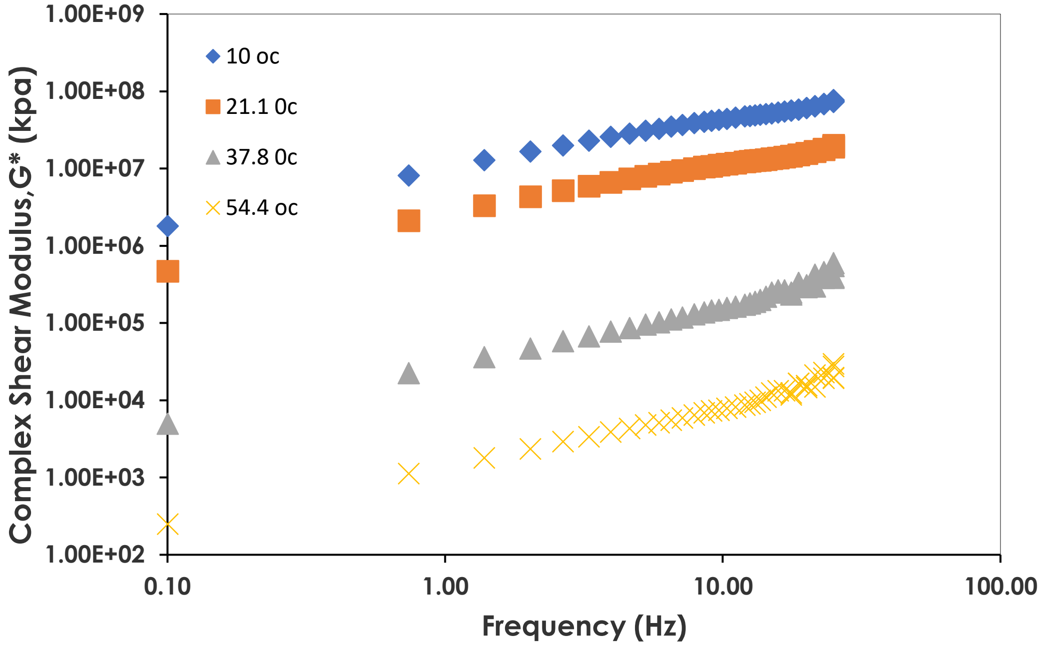



*Figure 4-6. Complex modulus verses frequency for aged 6% ANSS*

## 4.5 Master Curve

Generally, frequency sweep tests are performed in order to construct master curves that will determine the rheological properties of mixes. Dynamic shear complex moduli at different test temperatures and frequencies could be determined by using the time temperature superposition principle. In constructing the master curves using the time temperature superposition principle, test data collected from the DSR at different temperatures and loading times, in terms of stiffness (shear complex modulus & Phase angle), are compared to a reference temperature, which is in our case 21.1 °C. The data at any other temperatures were shifted with respect to time until various curves overlap almost perfectly to form a single master curve. Different scholars use different models for shifting to single reference temperatures. But a research developed at the University of Maryland showed that the master curve for binders can be represented by a sigmoidal function (Design Guide, 2004) defined by equation.

$$log(G^*) = \sigma - \frac{\alpha}{1 + e^{(\beta+\sigma \log(fr))}}$$

$$\Phi = - \ 90 * \sigma\alpha - \frac{exp^{(\beta+\sigma*log(fr))}}{[1+e^{(\beta+\sigma*log(fr)}]^2}$$

Where G* = dynamic modulus

fr = loading frequency at the reference temperature (reduced frequency)

Φ= phase angle

δ = minimum modulus value

δ + α = maximum modulus value

β, γ = parameters describing the shape of the sigmoidal function

In this research the master curves are constructed fitting a sigmoidal function to the measured complex modulus test data using nonlinear least squares regression, which can be done using the Solver Function in the Excel spreadsheet. The shifting could be done by solving shift factors simultaneously with the coefficients of the sigmoidal function, using any available shifting function to solve reduced frequency ($f_r$) as a function of temperature.

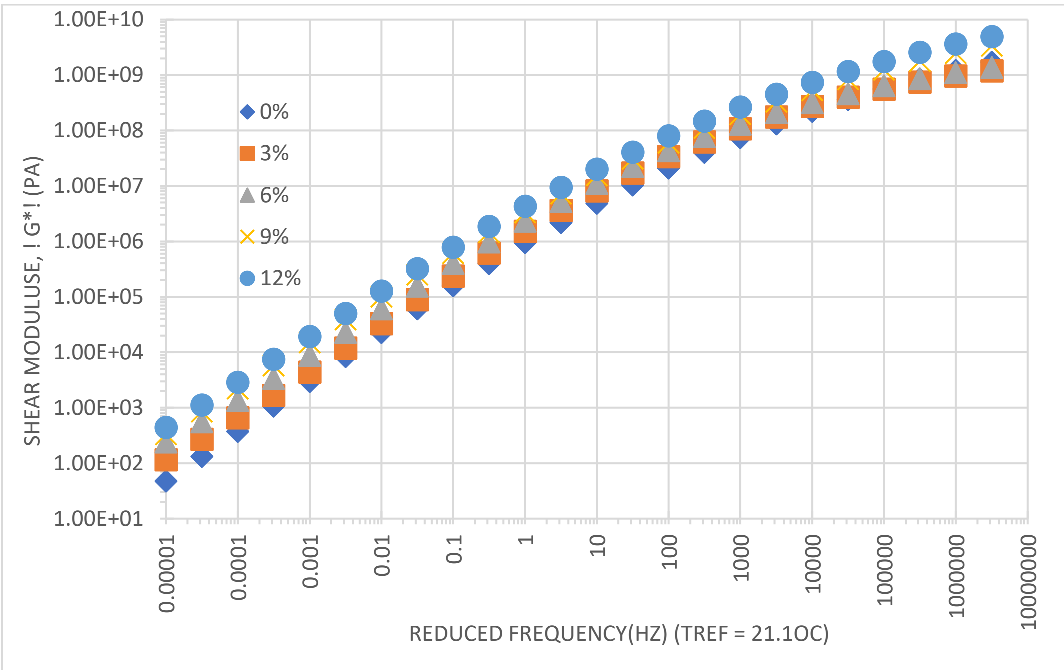
For complex modulus master curve, parameter γ influences the steepness of the function (rate of change between minimum and maximum) and β, the horizontal position of the turning point. Parameters β and γ, on the other hand, depend on the characteristics of the asphalt binder and the magnitude of δ and α (Design Guide, 2004).

Accordingly, the DSR data from the four test temperatures ($10^0$,21.1°, 37.8°, and 54.4°C) were used to construct the master curves for asphalt binder and asphalt binder containing ANSS both aged and unaged, and the following shift factors have been developed to construct the master curves for complex modulus.

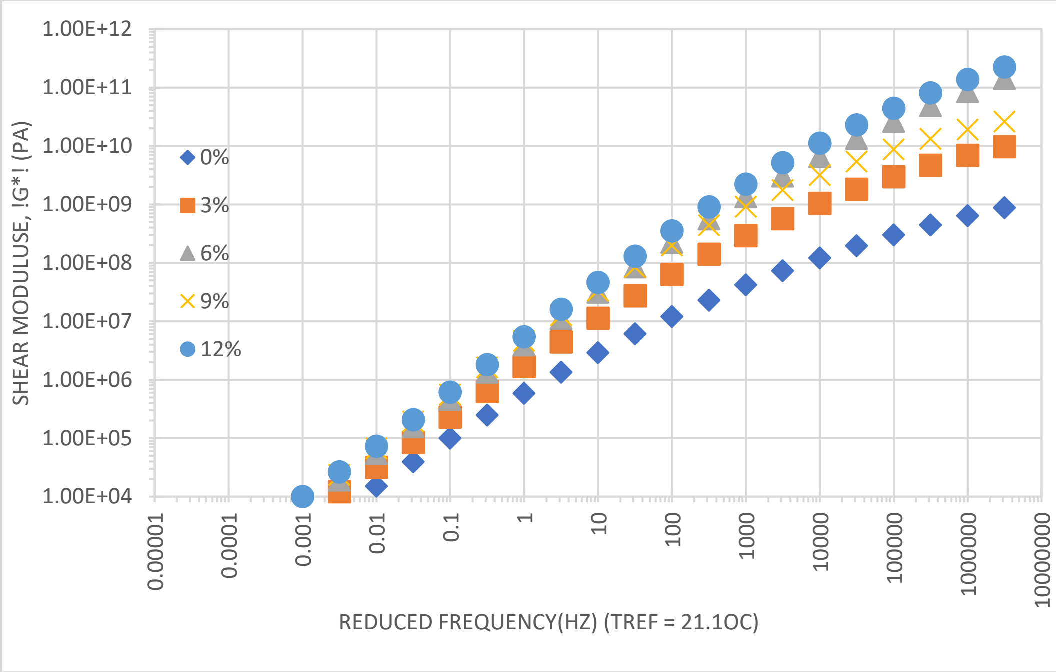
*Table 3. Shift factor for Complex Modulus Master Curves for aged and Unaged Binder*

| Condition | ANSS Content(%) | α | β | γ | δ | a10 | a21.1 | a37.8 | a54.4 |
|---|---|---|---|---|---|---|---|---|---|
| RTFO | 0 | 25.249 | -1.654 | -15.127 | 0.197 | 1.328 | 0.000 | -1.312 | -2.781 |
| | 3 | 19.723 | -0.129 | -4.034 | 0.173 | 0.712 | 0.000 | -1.769 | -3.136 |
| | 6 | 24.996 | -0.582 | -9.777 | 0.151 | 1.004 | 0.000 | -1.483 | -2.608 |
| | 9 | 23.163 | -0.535 | -7.931 | 0.155 | 0.747 | 0.000 | -1.795 | -3.073 |
| | 12 | 12.039 | -0.147 | -0.143 | 0.311 | 0.700 | 0.000 | -1.273 | -2.399 |
| Unaged | 0 | 15.182 | -0.884 | -4.974 | 0.233 | 0.939 | 0.000 | -1.462 | -2.683 |
| | 3 | 10.504 | -0.716 | -0.864 | 0.333 | 0.988 | 0.000 | -2.437 | -4.004 |
| | 6 | 10.582 | -0.763 | -0.850 | 0.317 | 0.990 | 0.000 | -2.445 | -4.014 |
| | 9 | 13.703 | -0.846 | -3.064 | 0.244 | 0.982 | 0.000 | -2.520 | -4.117 |
| | 12 | 13.874 | -0.805 | -2.902 | 0.240 | 0.970 | 0.000 | -2.508 | -4.110 |

The parameter α is defined as minimum stress level that would cause the damage; δ + α are defined as the maximum stress that would cause instantaneous damage; and the β and γ are described as the shape of the sigmoidal function. All of these values vary for each binder type. As for the temperature shift factors, $a_{21.1}$ is zero for all the binder types because all the parameters are shifted to 21.1°C. for $a_{10}$ is positive since it shifted to right of at temperature of 21.1 $^{0}$c Whereas $a_{37.8}$ and $a_{54.4}$ the values are all negative because the stiffness parameters are shifted to the left to reduced temperature which is 21.1°C.



(a)

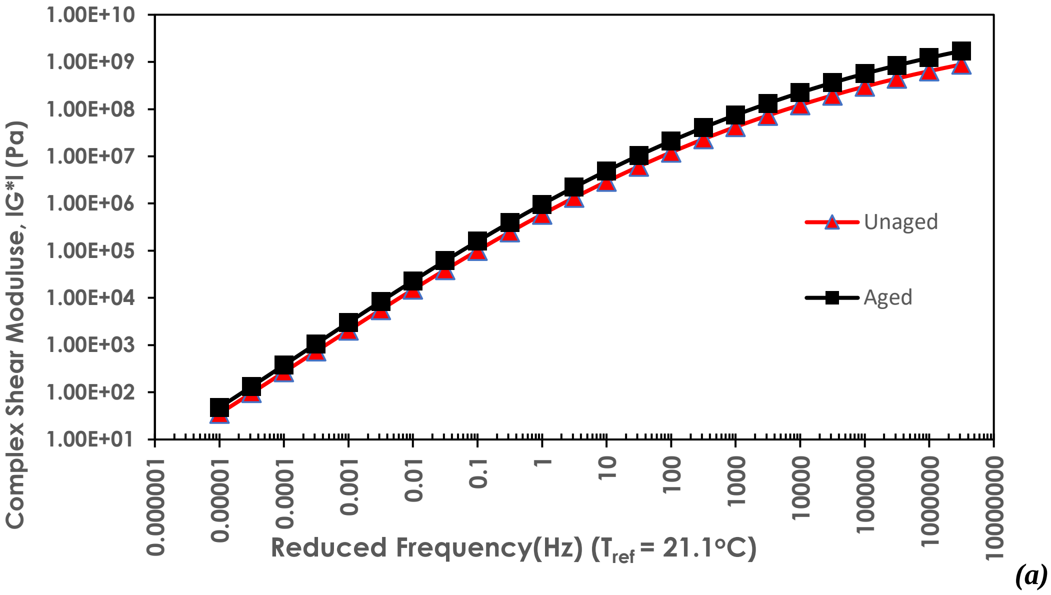



(b)

*Figure 4-7. Complex Modulus Master Curve (a) for unaged Binder (b) for Aged Binder*

Figure 4-7 (a) and (b) shows that the main rheological parameter for unaged and aged binder. complex shear modulus (G*) behaves in different ways as an effect of temperature, frequency, content of modifier and ageing. For all binders in almost similar pattern shear stiffness decreases as temperature increases. At low frequency and high temperature, the modulus increases appreciably as modifier increases showing better rutting resistance at this zone. Since frequency relates to loading rate (speed), from the graph the effect of modifier is evident at typically operating frequency/speed (0.01 to 10Hz).

From above we can summarize that the modifier appreciably improves the complex shear modulus of the virgin binder at higher temperatures. Rutting is a serious problem at high temperature due to slow moving traffic as observed from ANSS modifier improves the pavement performance against this distress by increasing the stiffness of the binder.



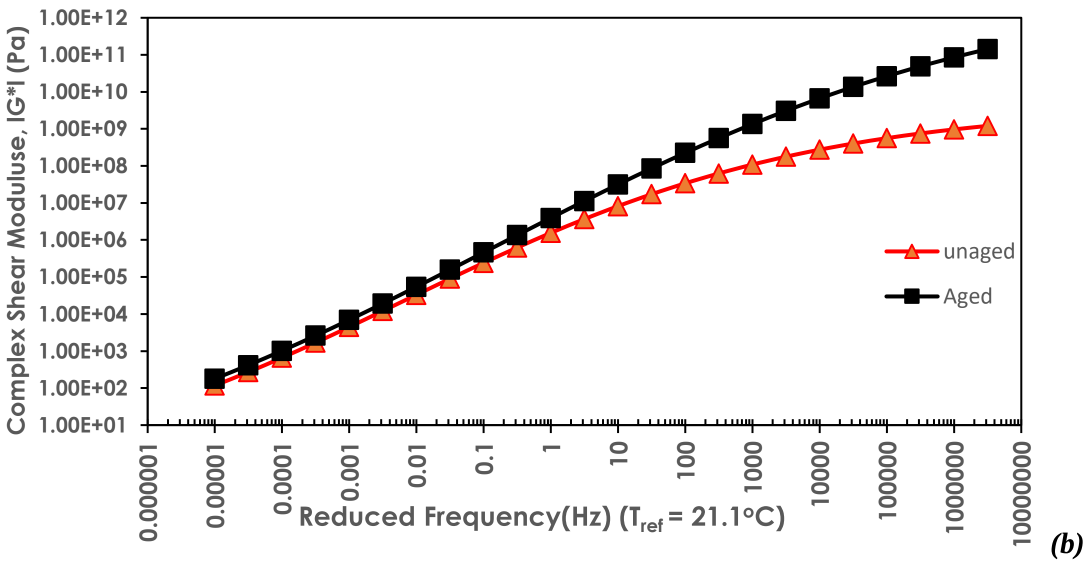




*Figure 4-8. Master Curve of (a) neat binder (b) 3% ANSS modified binder*

Figures 4-8 (a) and (b) clearly shows that the asphalt binder containing ANSS is less affected with aging compared to the virgin asphalt binder which is also an advantage for the mixed binder i.e. the presence of ANSS reduces the aging effect of a binder. This may be due to the fact that bitumen contains Sulphur and as the ANSS content increases the percentage of bitumen decreases there by decreasing sulfur content. John mentioned that sulfur plays an important role in the ageing of bitumen because it is more chemically reactive than hydrogen or carbon, and can oxidizes more easily than the hydrocarbons.

## 4.1 Phase Angle Master Curve

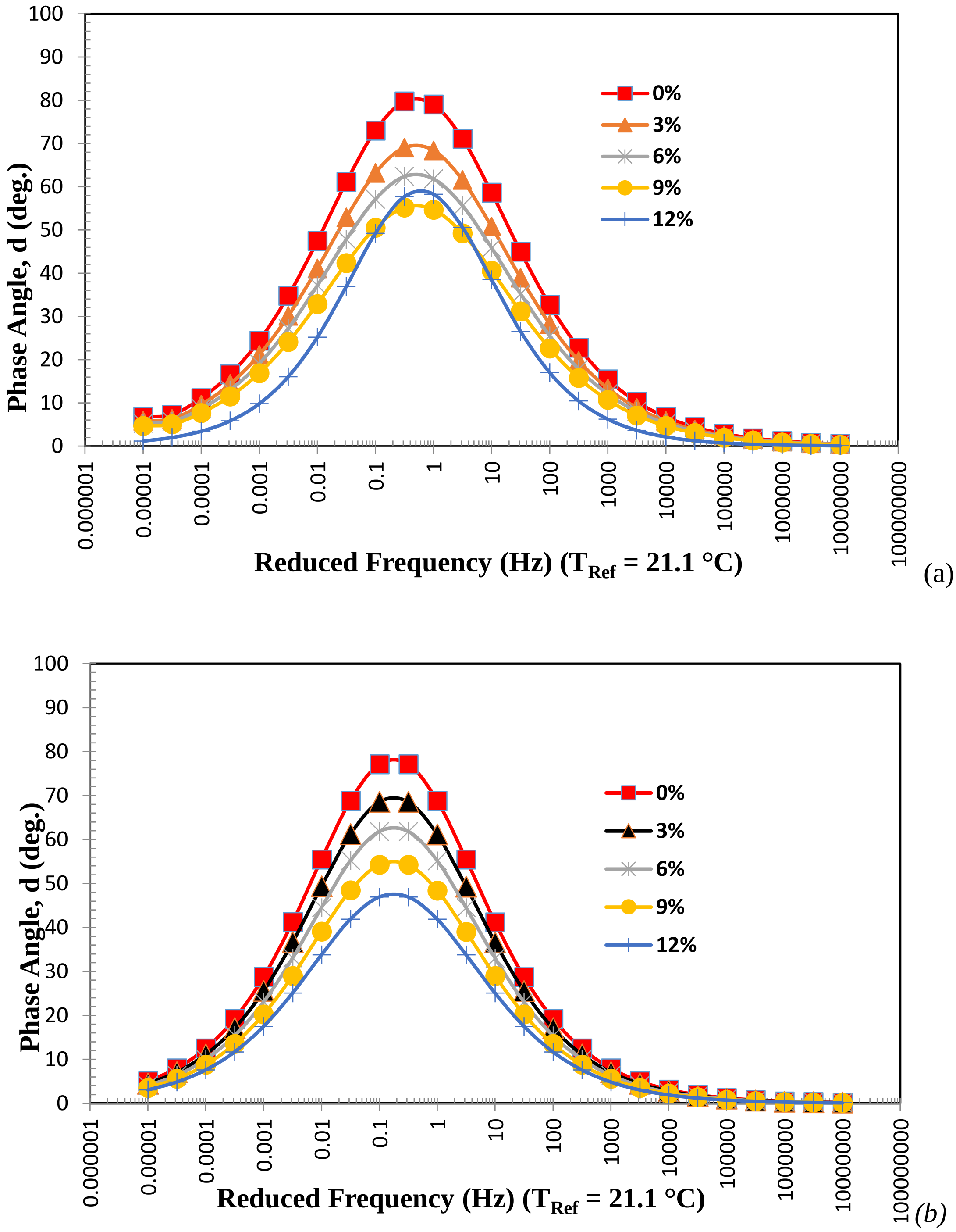



*Figure 4-9. Phase angle (a) for unaged Binder (b) for Aged Binder*

Figure 4-9 (a) and (b) present a comparison of unaged and aged ANSS binder mixture to phase angle relationships. three conclusions can be drawn from this figure.

Firstly, at low frequency and phase angles (≤ 45º) for both aged and unaged mixes similar relationships exist between ANSS asphalt mixtures. This can be attributed to dependency of asphalt mixture properties at low temperatures on bitumen properties and the ANSS proportions of the asphalt mixture. However, there are variations in the asphalt mixture-bitumen phase angle relationships between the different mixtures at high bitumen phase angles (> 45º).

Secondly, asphalt mixture phase angles start to decrease at bitumen phase angle values between 55 and 70º as the content of ANSS increase. This corresponds to the increased influence of the ANSS mix on asphalt mixture properties at high temperatures and low frequencies resulting in increasing elastic behavior of the asphalt mixtures.

Finally, as content of modifier increases a decrease in phase angle at high temperature and low frequency applicably. but, at low temperature and high frequency addition of ANSS in asphalt binder has almost no effect.

## 4.2 Statistical Analysis of FST Result Using ANOVA

Succeeding the laboratory procedures and data collection, statistical analysis was performed to evaluate the significance of addition of ANSS to asphalt binder using one-way analysis of variance (ANOVA).

A one-way ANOVA uses two different estimates of sample variance. The first estimate is called the between-group variance, and it involves finding the variance of the means. The second estimate, the within-group variance, is made by computing the variance using all the data and is not affected by differences in the means.

Master curve were used at frequency of 0.1 ,10 and 25 HZ, If the F value determined to be smaller than the F crit, then the factors are statistically significance and vice versa. In this study, ANOVA is calculated using Excel at significance level of 0.1.

The five groups of independent variables i.e. 0%, 3%, 6% and 9%, and 12 % were considered. On this research the ANOVA consists of two random samples from each of five independent groups. And the null hypothesis (Ho) is that the neat asphalt binder and the three percentages of ANSS are equally effective. This means, there is no rheological behavior change in asphalt binder up on addition of different percentages of ANSS. Therefore, the null hypothesis Ho: $\mu 1 = \mu 2 = \mu 3 = \mu 4 = \mu 5$. Whereas, the alternative hypothesis H1: at least one percentage has a change in rheological property of asphalt binder up on addition of ANSS

The test statistics for one-way ANOVA is the F ratio i.e. the ratio of between the group variance and the within group variance. For each of the five percentages two samples were tested.

From the ANOVA the results obtained from excel are summarized and presented in the table below. Details of ANOVA hypothesis testing is presented in Appendix D of the research paper.

*Table 4. Summary of a hypothesis test of Master Curve*

| Frequency (HZ) | F-Value | P-Value | F-Critical | Decision |
|---|---|---|---|---|
| 0.1 | 5.54 | 0.04 | 11.39 | Accept |
| 10 | 8.13 | 0.02 | 11.39 | Accept |
| 25 | 81.49 | 0.00 | 11.39 | Reject |

Table 4 illustrates that the decision to accept or reject the null hypothesis is made by comparing the test statistics computed F with the critical value. If the computed F value exceeds the critical value, the hypothesis is rejected; if not, the hypothesis is not rejected. The ANOVA result for FST i.e. Master Curve, indicates that the F value exceeds the critical value for frequency of 25 at 0.1 level of significance. While for frequency of 0.1 and 10 Hz is the reverse, Therefore, the null hypothesis is rejected for frequency of 25 and accepted for frequency of 0.1 and 10Hz. Hence this research accepted that addition of ANSS on asphalt binder affects the rheological property of asphalt binder at low frequency (high temperature) and intermediate frequency (moderate temperature) while it doesn’t affect it at high frequency (low temperature).

## 4.7 Multiple Stress Creep Recovery (MCSR) Test

The analysis is carried out considering the main purposes of MSCR test. Rutting prediction, indication of elastic response and specification preparation of a binder are the main purposes.

MSCR test was conducted after the determination of the performance grade of each sample. The PG is then used to establish the MSCR test temperatures as organized in table below.

*Table 5. MSCR Test Temperature Based on PG*

| Percentage of ANSS | Before RTFO | After RTFO | Performance Grade (AASHTO M320) |
|---|---|---|---|
| 0 | 70 | 64 | PG 64-xx |
| 3 | 64 | 58 | PG 58-xx |
| 6 | 64 | 64 | PG 64-xx |
| 9 | 64 | 64 | PG 64-xx |
| 12 | 64 | 64 | PG 64-xx |

For this study, repeated shear creep testing was conducted at three temperatures 52°, 58°, and 64° after determination of performance grade for each binder mix. Test Effect for Performance Grade determination is presented on Appendix E.

The MSCR test Effect (software output) contains huge data to represent in tables here. Therefore, as an illustration for only 6 % ANSS binder mix test Effect is organized graphically just for illustration as shown in figure below. See Appendix F for all other ANSS Binder Mix.

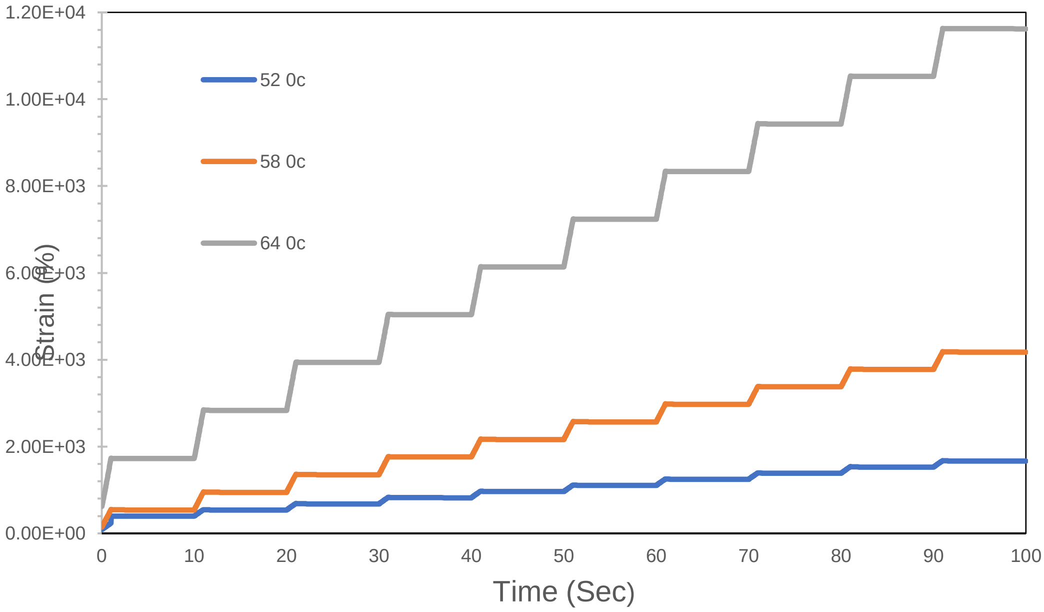
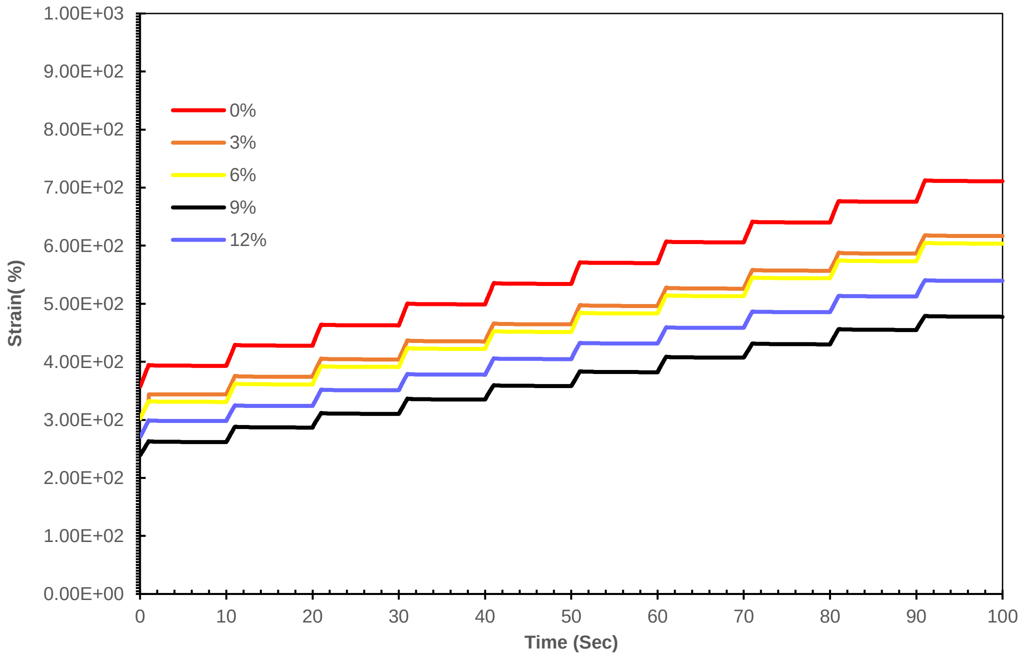
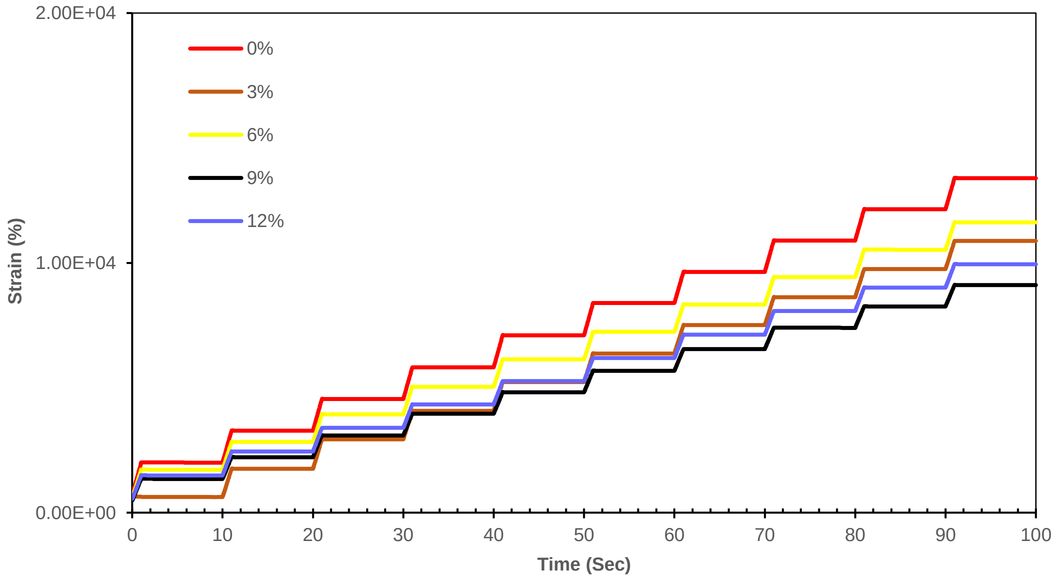


*Figure 4-10. MSCR Graph for 6 % ANSS Binder Mixes*




(a)




(b)

*Figure 4-11. Effect of ANSS on Strain (a) at 0.1 KPa ($64^0$c) and (b) at 3.2 Kpa ($64^0$c)*

Figures above show that the total strain was influenced by addition of ANSS. As the temperature increase the total strain value increase as expected since at high temperature the binder becomes viscous. And as the ANSS percentage increases up to 9 % the total strain value decreases and slightly increases for the 12 % showing the mix improves up to 9 % of ANSS.

To evaluate the main parameters Non-recoverable creep compliance (Jnr) is calculated as non-recoverable strain divided by applied stress (0.1kpa or 3.2kPa). The calculated values are organized using separate table for each type of binder as follows. Test Effect for Non-recoverable creep compliance (Jnr) is presented on Appendix F.

*Table 6. Summary of Jnr value for different percentage of ANSS*

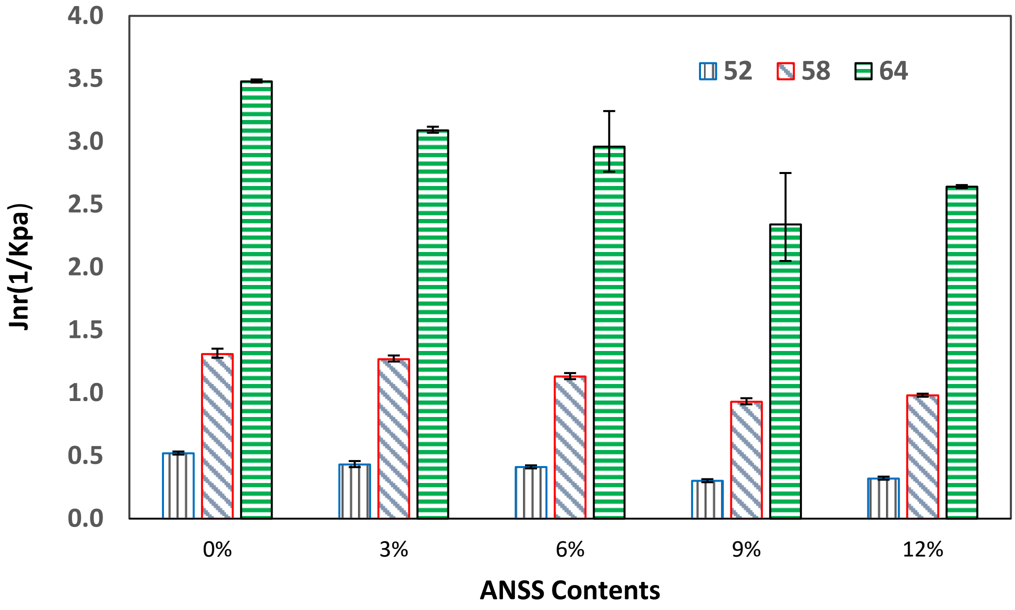
| | | Temperature ($^0$C) | | |
|---|---|---|---|---|
| ANSS (%) | JNR @ | 52 | 58 | 64 |
| 0 | 100 | 0.52 | 1.31 | 3.48 |
| | 3,200 | 0.63 | 1.45 | 3.90 |
| 3 | 100 | 0.43 | 1.27 | 3.09 |
| | 3,200 | 0.54 | 1.36 | 3.15 |
| 6 | 100 | 0.41 | 1.13 | 2.96 |
| | 3,200 | 0.48 | 1.24 | 3.39 |
| 9 | 100 | 0.30 | 0.93 | 2.34 |
| | 3,200 | 0.44 | 0.99 | 2.65 |
| 12 | 100 | 0.32 | 0.98 | 2.64 |
| | 3,200 | 0.52 | 1.09 | 2.89 |

Now it is possible to evaluate all the binders by contrasting the calculated basic MSCR parameters with respect to the limits of those parameters described under standard specification for performance graded asphalt binder using MSCR test, AASHTO M 322.

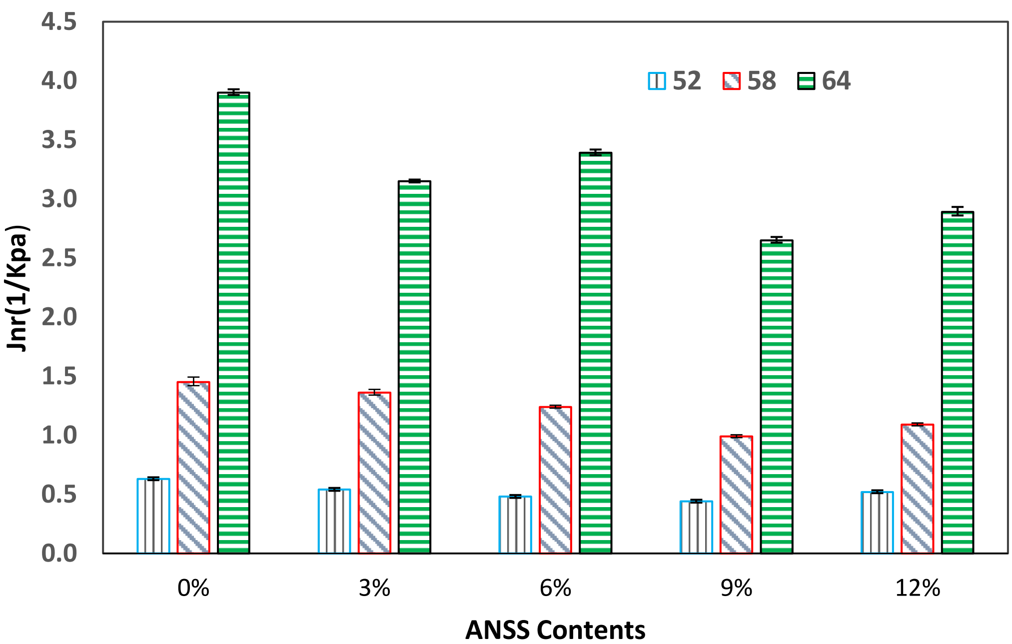
*Table 7. Binder Specification Requirement Based on MSCR test*

| **Traffic Designation** | **Traffic level (ESALs)** | **Load Rate** | **Jnr $_{3.2}$, Max kPa$^{-1}$** |
|---|---|---|---|
| Standard Traffic "S" | <10 million | >70 km/h | 4.5 |
| Heavy Traffic "H" | 10 to 30 million | 20 to 70 km/h | 2 |
| Very Heavy Traffic "V" | >30 million | <20 km/h | 1 |
| [a] Extremely Heavy Traffic "E" | >30 million | <20 km/h | 0.5 |

The non-recoverable creep compliance (Jnr) at 3.2kPa is the fundamental parameter to evaluate rutting potential. To examine this, the Jnr determined for both neat and modified binders were graphically represented below.




(a)




(b)

*Figure 4-12. MSCR test result, $J_{nr}$ value at different loading (a) 100 Pa (b) 3200 Pa*

From the figures 4-12 (a) and (b) illustrate that the rutting parameter (Jnr) decreases as the percentage of ANSS increase and increases as the temperature increases. This refers that addition of ANSS could improve the resistance of asphalt pavements to rutting at all temperatures especially at lower temperatures. Table 7 shows the rutting parameter (Jnr) for all the binder mixes. Other MSCR test Effects are shown in **Appendix F** of the research paper.

## 4.3 Summary

Following the laboratory test presented this chapter summarized the result in following manner at first, the penetration value decreases, the softening point and flash and fire point are increase, the ductility don't show any change as the ANSS content increased which show that in comparison with the control mix value of empirical or traditional test is generally high. nevertheless, further injection of ANSS into the mixture lead to a decrease in value of the test. This is because result shows excessive application of ANSS may decrease contact point in the mixture. Secondly the linear viscoelastic range (LVER) of the binder decreases as the ANSS content increased showing ANSS has effect on rheological property which implies ANSS has improve rheological property of the mix. inaddition at high temperature and low loading condition the stiffness of the binder increased as the percentage of ANSS increased which shows addition of ANSS improves asphalt binder properties at high temperatures. Finally, the strain value and the rutting parameter Jnr decreased as the ANSS content increased which implies addition of ANSS in asphalt binder reduces rutting of pavements.

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# CHAPTER 5: CONCLUSION AND RECOMMENDATION

## 5.1. Conclusion

After the introduction to the problem, a review of the literature, and analysis and discussion of the findings, this section intends to summaries the overall conclusions achieved through this research. Therefore, the significant findings of the current study are as follows.

1. The result shows that test result obtained from FST, the master curve improves behavior for asphalt binder up on addition of ANSS. Implies addition of ANSS on asphalt binder increases the stiffening property of asphalt binder at high temperatures (low loading frequencies).
2. In addition, MSCR test result reflected, the smallest total strain value was obtained for ANSS of 9%, followed by 12 %, 6% and 3% ANSS. Therefore, addition of ANSS improves the resistance of asphalt pavements to rutting.
3. Furthermore, Neat asphalt binder was more affected by aging compared to asphalt binder containing ANSS. Meaning addition of ANSS to asphalt binder decreases the aging effect of HMA mixtures.
4. Addition of ANSS has appreciatively positive result in conventional test.

In general, the aim this research which is addition ANSS to asphalt binder was to evaluate Effect of ANSS modified bitumen on asphalt performance. Therefore, it can be concluded that it is possible to partially replace asphalt binder with ANSS for wearing coarse up to 9% at dry areas.

## 5.2. Recommendation

Based on the study results the following recommendations are made.

- ANSS can be used as partial replacer for asphalt bitumen.
- It may be possible to use relatively lower asphalt bitumen when ANSS is used.
- 9% ANSS can be used to partially replace asphalt binder for a binder course at dry areas and for wet areas the same percentage could be used with addition of antistripping agents (adheres).
- It would be better if Ethiopian Roads Authority support and encourage ANSS modified related pavement technology considering its advantage to minimize pavement distress due to rutting in Ethiopia.
- It is also advisable if Ethiopian Roads Authority and other stake holders in the area of road construction show interest to use the latest binder specification system using Multiple Stress Creep Recovery test (AASHTO M 332) which is the new method that best relates with rutting performance. Moreover, it is because binder characterization has to be considering specific local condition of loading, temperature and others.

## 5.3. Future Study

There is always the opportunity for future research in the area of asphalt binders and mix characterization. For this reason, future research work may include:

- Further studies are needed to characterize binders composed of ANSS and asphalt binder using different grade bitumen.
- Further studies are needed to characterize the chemistry of binders composed of ANSS and asphalt binder.
- Further studies are needed to evaluate long term aging and low temperature effect using PAV and BBR composed of ANSS and asphalt binder.

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# APPENDIX-A CONVENTIONAL TEST RESULT

*Table 8. Empirical test of virgin and modified binders*

| Type of Test | Test Trials | ANSS IN % | | | | |
|---|---|---|---|---|---|---|
| | | 0% | 3% | 6% | 9% | 12% |
| Penetration at 25℃ , 100g,5s (0.1mm) | 1 | 68.60 | 67.5 | 65.8 | 65.3 | 65.3 |
| | 2 | 69.40 | 68.3 | 67.3 | 64.3 | 64.3 |
| | 3 | 67.10 | 68.3 | 68.3 | 64.2 | 65.3 |
| | **Average** | **68.4** | **68.0** | **67.1** | **64.6** | **65.0** |
| | 1 | 68.4 | 68.4 | 68.3 | 65.3 | 65.5 |
| | 2 | 69.5 | 67.3 | 67.3 | 65.2 | 64.2 |
| | 3 | 67.4 | 67.3 | 67.3 | 64.3 | 65.7 |
| | **Average** | **68.4** | **67.7** | **67.6** | **64.9** | **65.1** |
| | 1 | 68.6 | 66.4 | 65.4 | 65.3 | 64.8 |
| | 2 | 67.8 | 64.3 | 64.3 | 64.5 | 65.4 |
| | 3 | 69.2 | 68.3 | 64.3 | 65.3 | 66.3 |
| | **Average** | **68.5** | **66.3** | **64.7** | **65.0** | **65.5** |
| | **Overall Average** | **68.4** | **67.3** | **66.5** | **64.9** | **65.2** |
| | | **0.9** | **1.3** | **1.6** | **0.5** | **0.7** |
| | Brucket-1 | 155 | 155 | 155 | 155 | 155 |

| | | | | | | |
|---|---|---|---|---|---|---|
| Ductility at 25°C(mm) | Brucket-2 | 155 | 155 | 155 | 155 | 155 |
| | Brucket-3 | 155 | 155 | 155 | 155 | 155 |
| | Avarage | **155.0** | **155.0** | **155.0** | **155.0** | **155.0** |
| | Brucket-1 | 155 | 155 | 155 | 155 | 155 |
| | Brucket-2 | 155 | 155 | 155 | 155 | 155 |
| | Brucket-3 | 155 | 155 | 155 | 155 | 155 |
| | Avarage | **155.0** | **155.0** | **155.0** | **155.0** | **155.0** |
| | Brucket-1 | 155 | 155 | 155 | 155 | 155 |
| | Brucket-2 | 155 | 155 | 155 | 155 | 155 |
| | Brucket-3 | 155 | 155 | 155 | 155 | 155 |
| | Avarage | **155.0** | **155.0** | **155.0** | **155.0** | **155.0** |
| | **Overall Average** | **155.0** | **155.0** | **155.0** | **155.0** | **155.0** |
| Softening point (°C) | Ring-1 | 51.50 | 51.00 | 52.50 | 52.90 | 52.20 |
| | Ring-2 | 50.50 | 52.00 | 51.50 | 51.70 | 51.80 |
| | Average | **51.00** | **51.50** | **52.00** | **52.30** | **52.00** |
| | Ring-1 | 50.3 | 51.2 | 52.2 | 52.3 | 52.2 |
| | Ring-2 | 51.4 | 51.3 | 51.3 | 52.8 | 52.1 |
| | Average | **50.85** | **51.25** | **51.75** | **52.55** | **52.15** |
| | Ring-1 | 50.3 | 51.2 | 51.2 | 52.4 | 51.3 |
| | Ring-2 | 51.1 | 50.8 | 51.3 | 52.2 | 52.1 |

| | | | | | | |
|---|---|---|---|---|---|---|
| | Average | **50.70** | **51.00** | **51.25** | **52.30** | **51.70** |
| | **Overall Average** | **51.2** | **51.1** | **51.7** | **52.7** | **52.0** |
| Flash & Fire Point (°C) | Trial -1 | 307 | 310 | 314 | 325 | 318 |
| | Trial -2 | 305 | 306 | 310 | 319 | 322 |
| | Average | **306.00** | **308.00** | **312.00** | **322.00** | **320.00** |
| | Trial -1 | 305 | 308 | 312 | 323 | 316 |
| | Trial -2 | 304 | 305 | 308 | 317 | 319 |
| | Average | **304.50** | **306.50** | **310.00** | **320.00** | **317.50** |
| | Trial -1 | 305 | 307 | 311 | 321 | 317 |
| | Trial -2 | 304 | 306 | 309 | 318 | 319 |
| | Average | **304.50** | **306.50** | **310.00** | **319.50** | **318.00** |
| | **Overall Average** | **305.00** | **307.00** | **310.67** | **320.50** | **318.50** |

# APPENDIX-B EFFECT OF TEMPERATURE OF AST

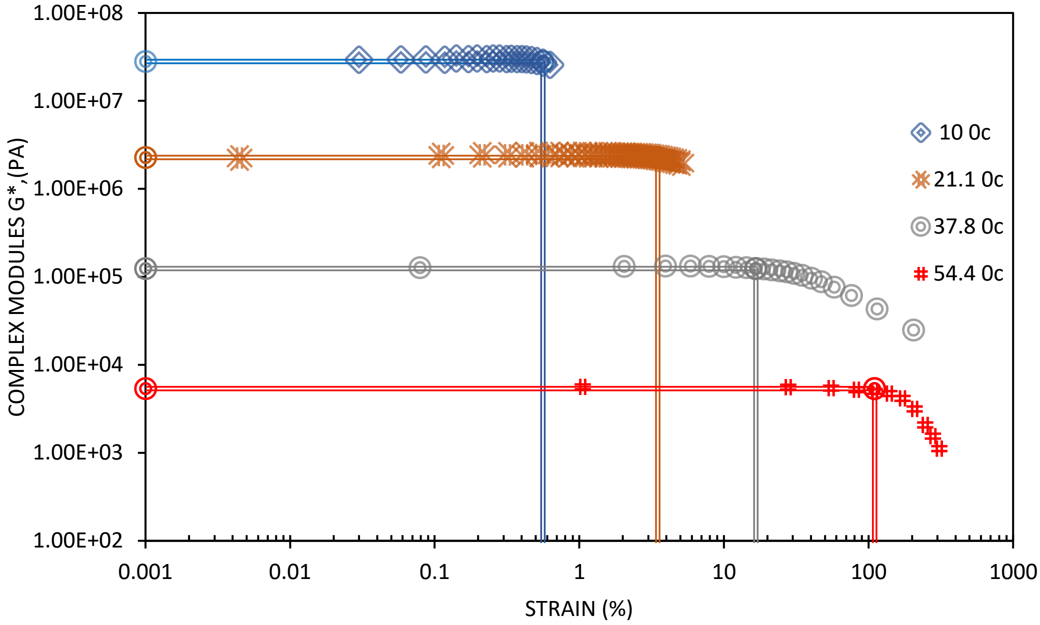


*Figure B-1. Effect of temperature on original Binder*

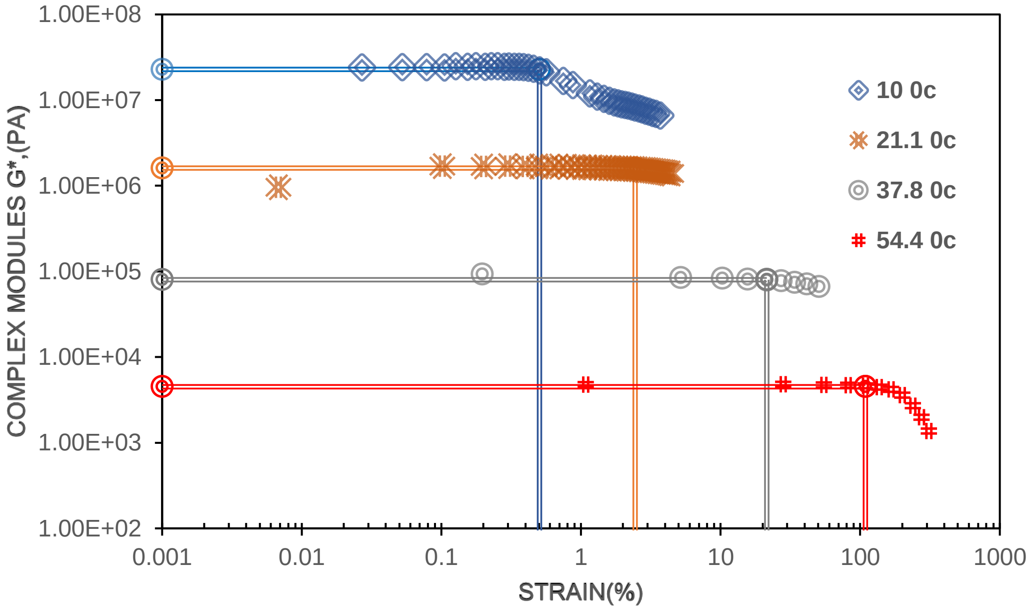


*Figure B-2. Effect of Temperature on 3 % ANSS Unaged Binder*

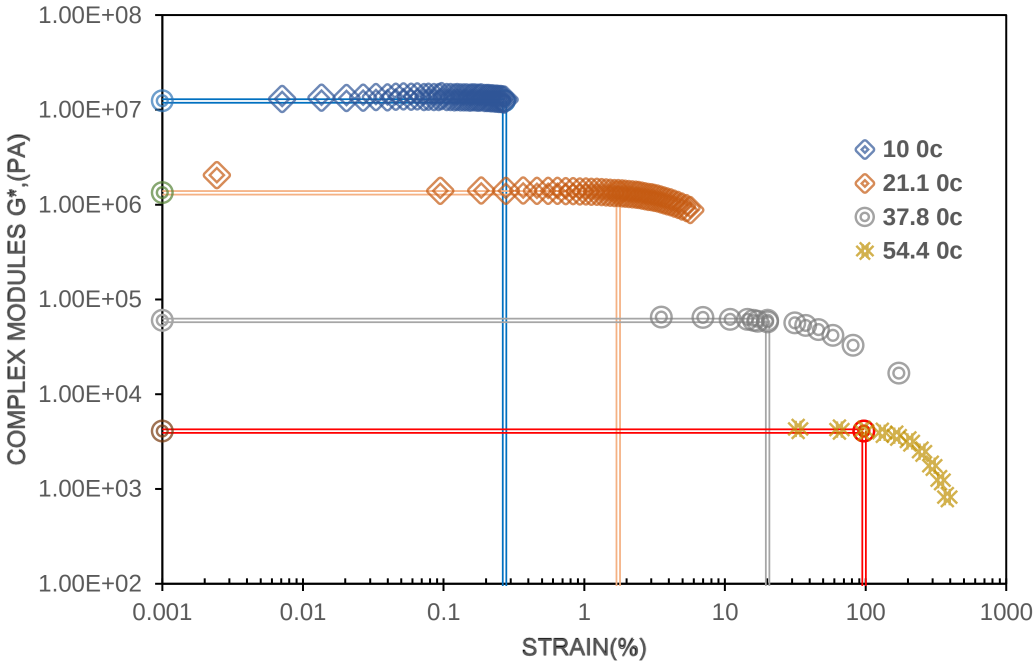


*Figure B-3. Effect of Temperature on 6 % ANSS Unaged Binder*

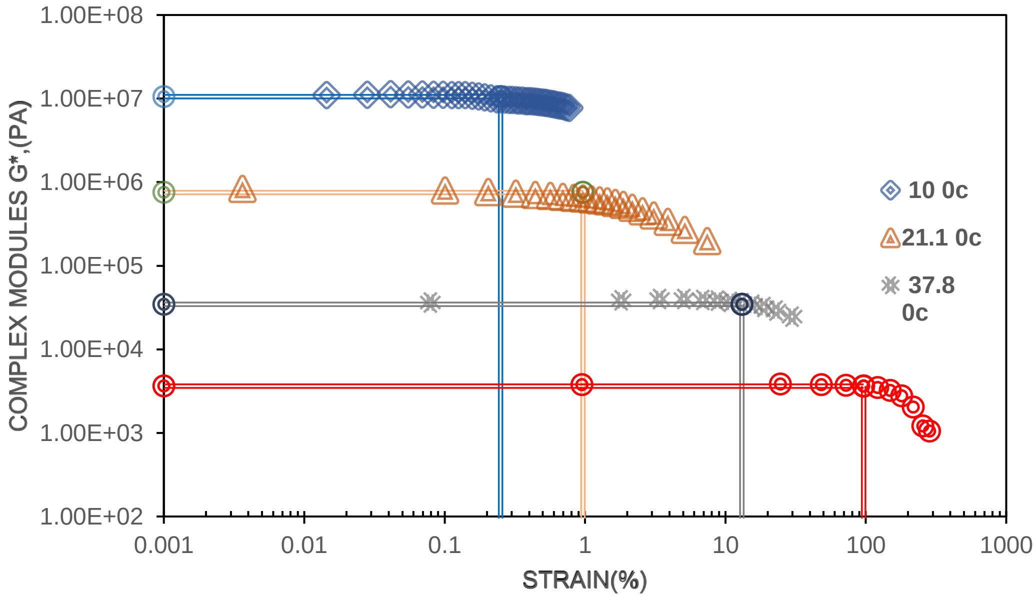


*Figure B-4. Effect of Temperature on 9 % ANSS Unaged Bin*

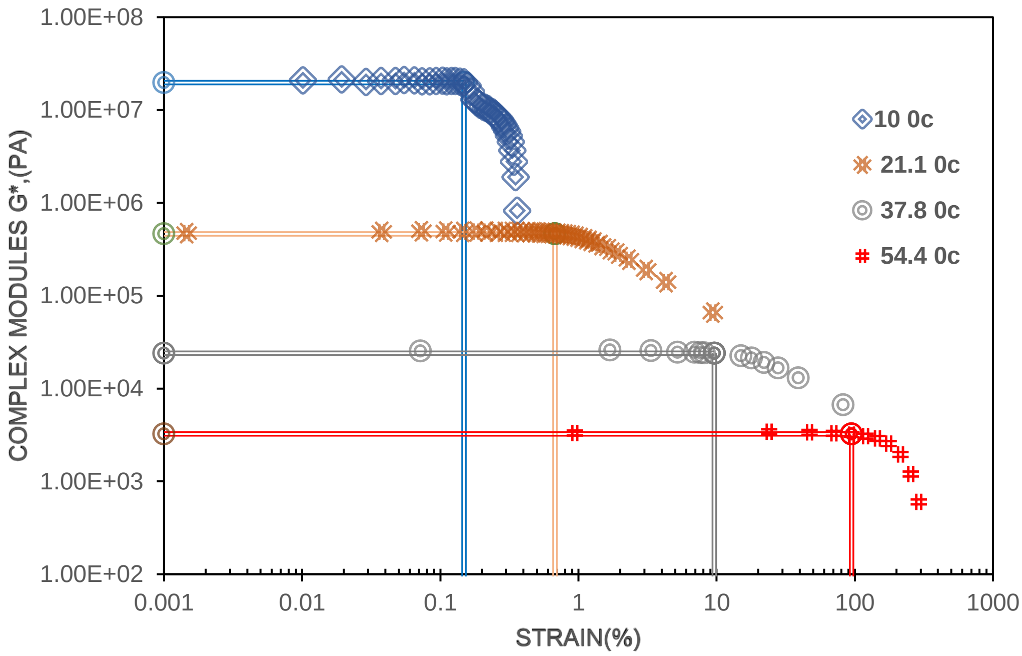


*Figure B-5. Effect of Temperature on 12 % ANSS Unaged Binder*

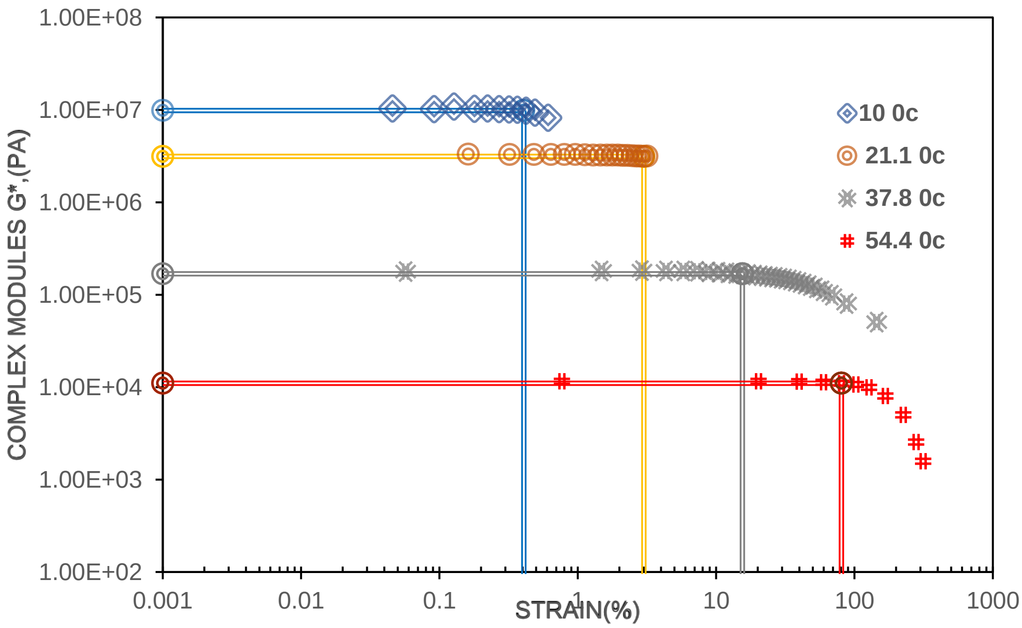


*Figure B-6. Effect of temperature on 0 % Aged Binder*

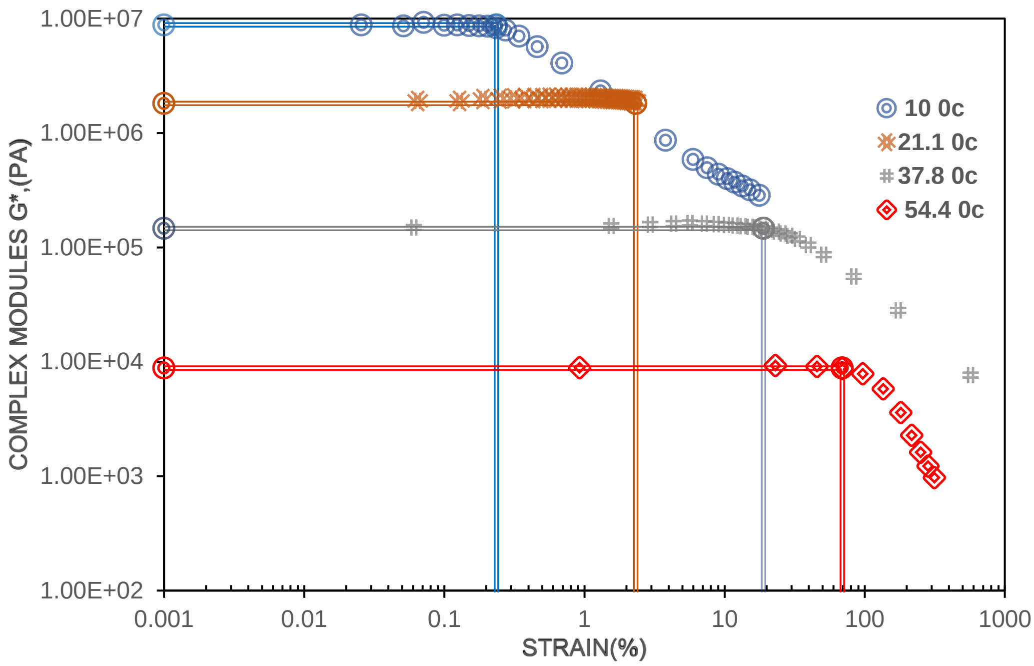


*Figure B-7. Effect of temperature on 3 % Aged Binder*

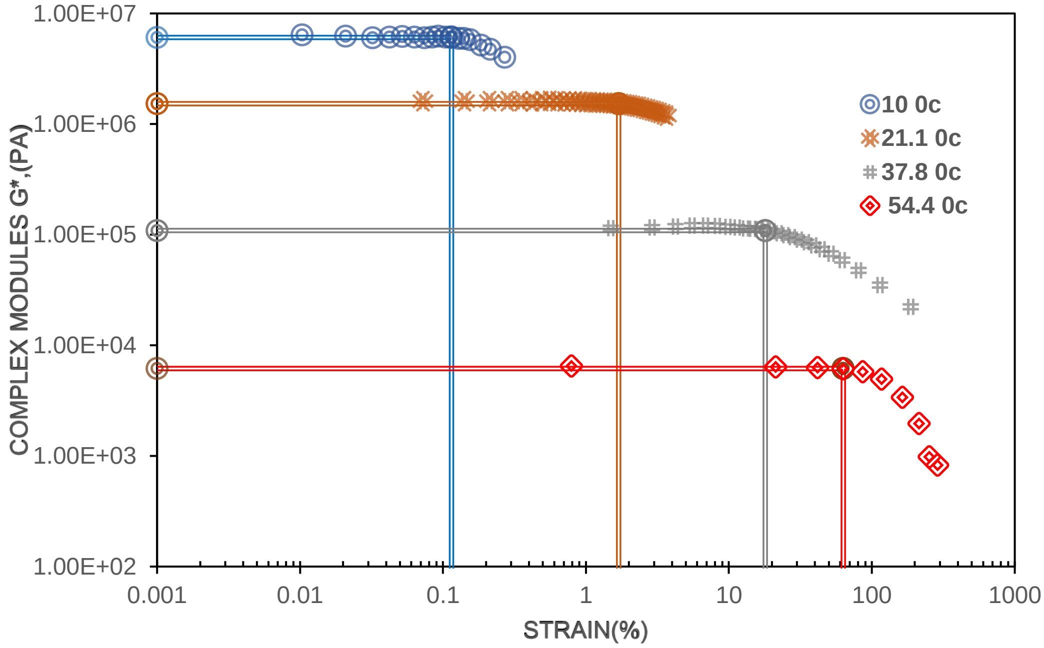


*Figure B-8. Effect of temperature on 6% Aged Binder*

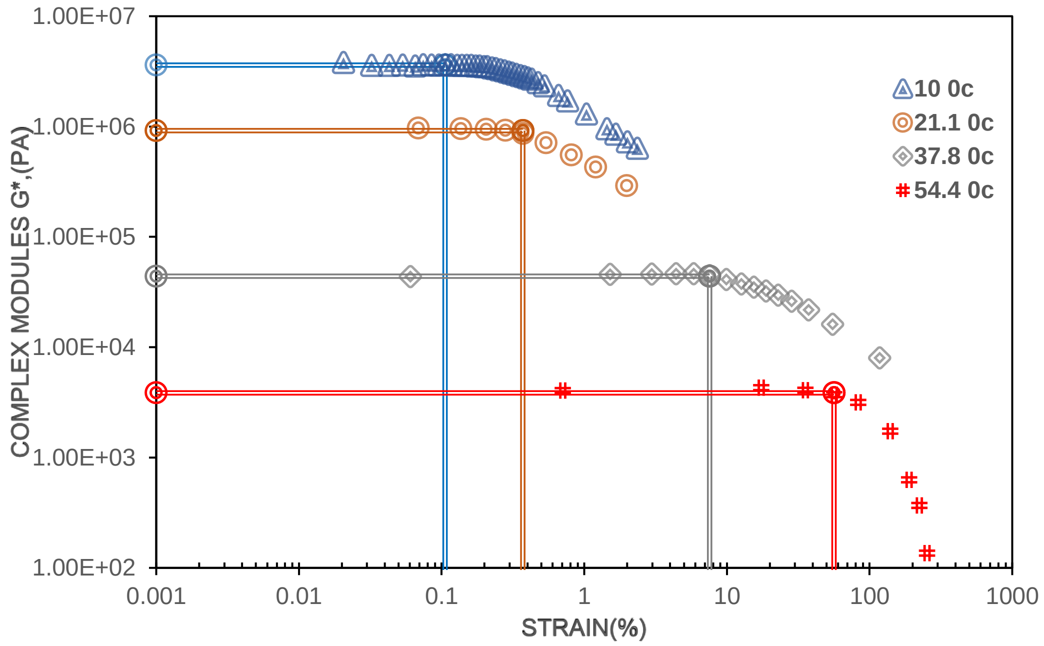


*Figure B-9. Effect of temperature on 9% Aged Binder*

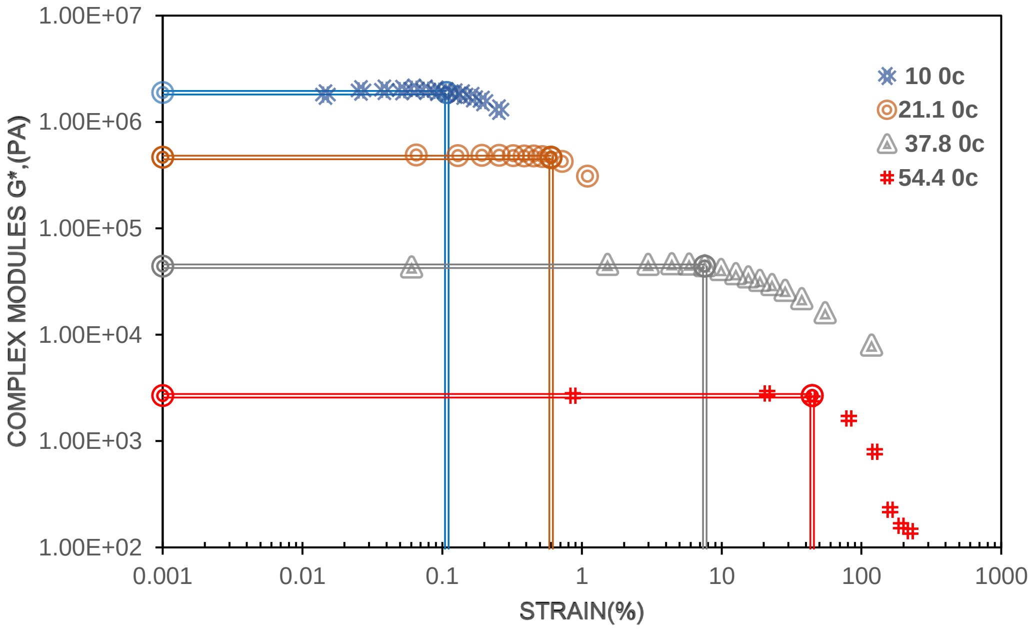


*Figure B-10. Effect of temperature on 12 % Aged Binder*

# APPENDIX-C FREQUENCY SWEEP TEST RESULT

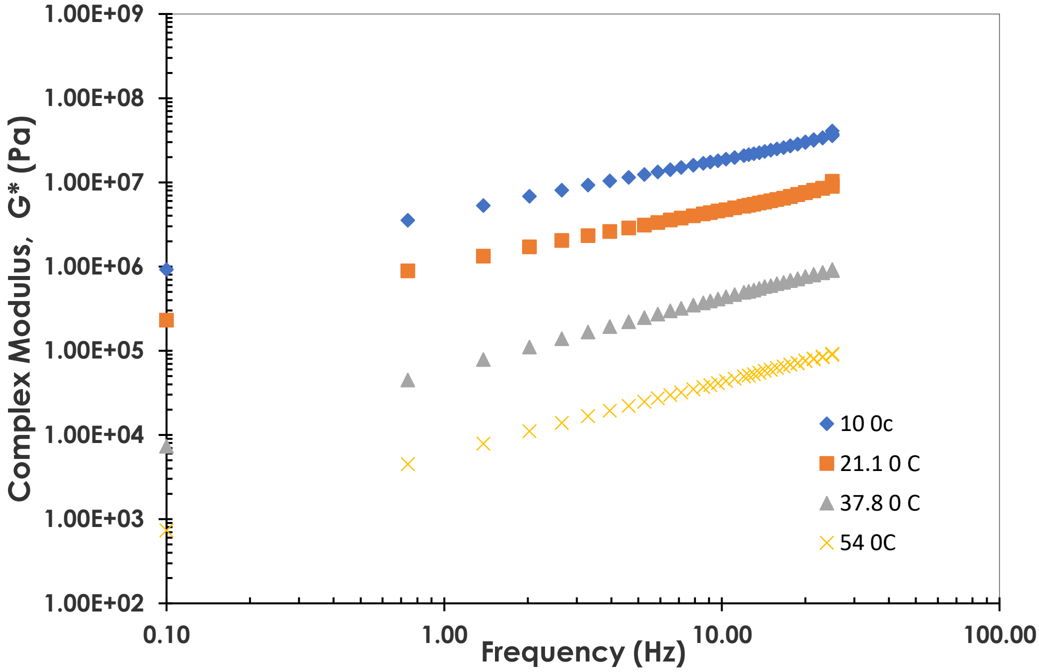


*Figure C-1. Unaged neat Binder*

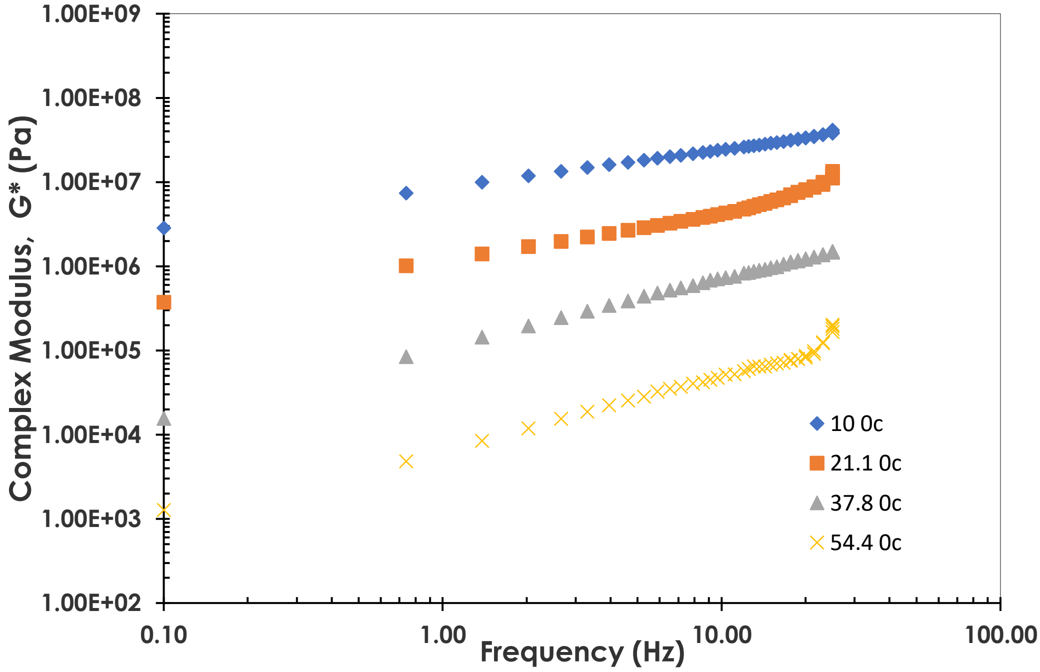


*Figure C-2. aged neat Binder*

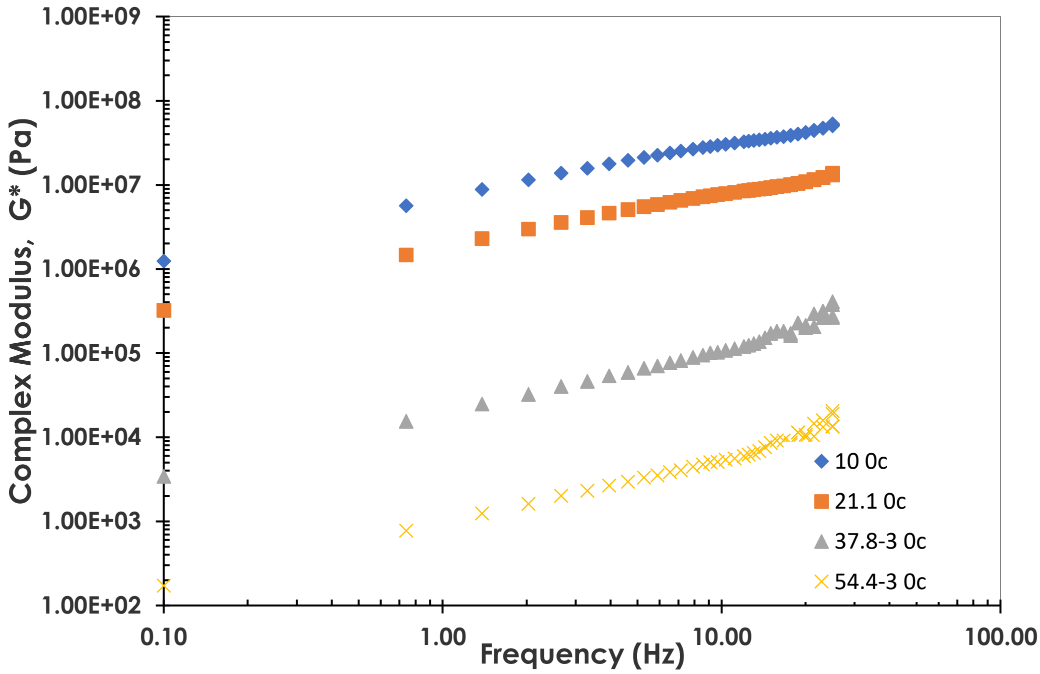


*Figure C-3. 3% Unaged Binder*

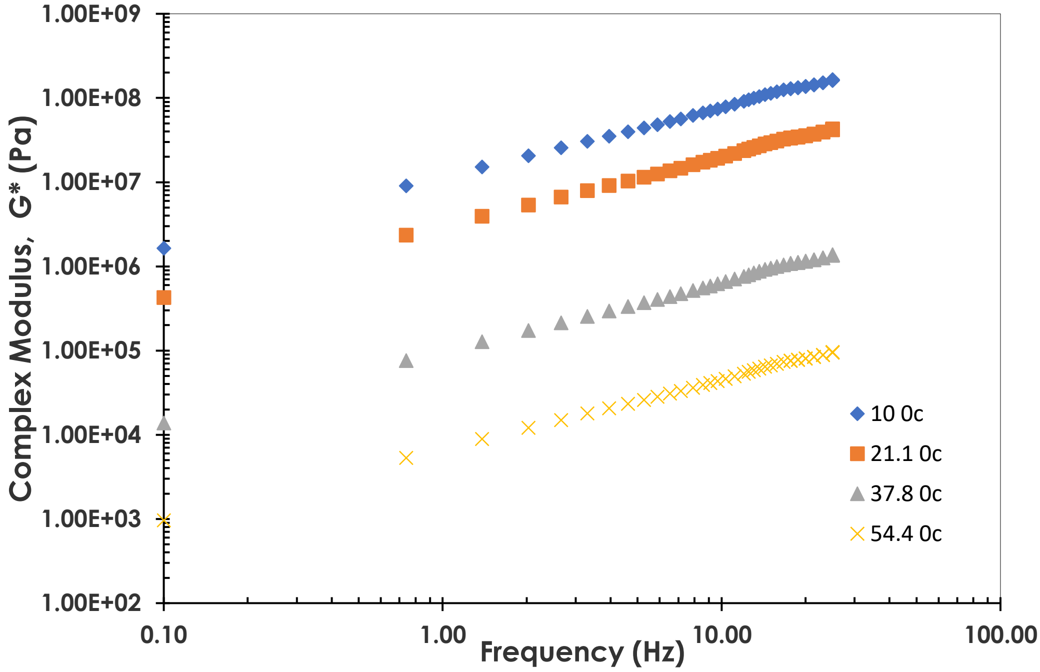


*Figure C-4. 3 % aged Binder*

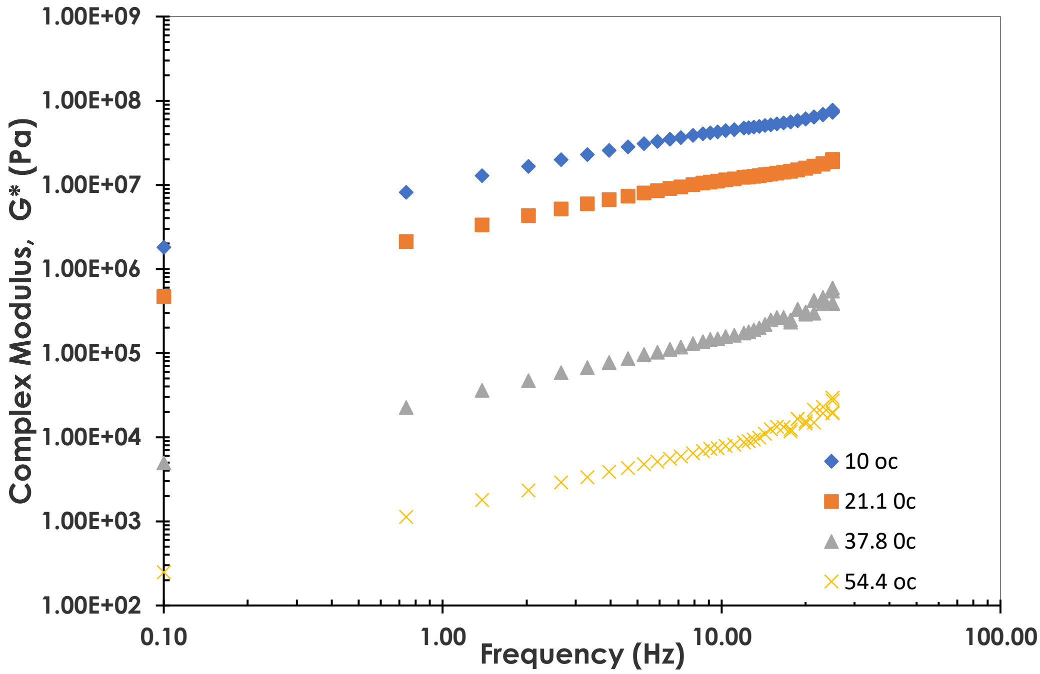


*Figure C-5. 6 % Unaged Binder*

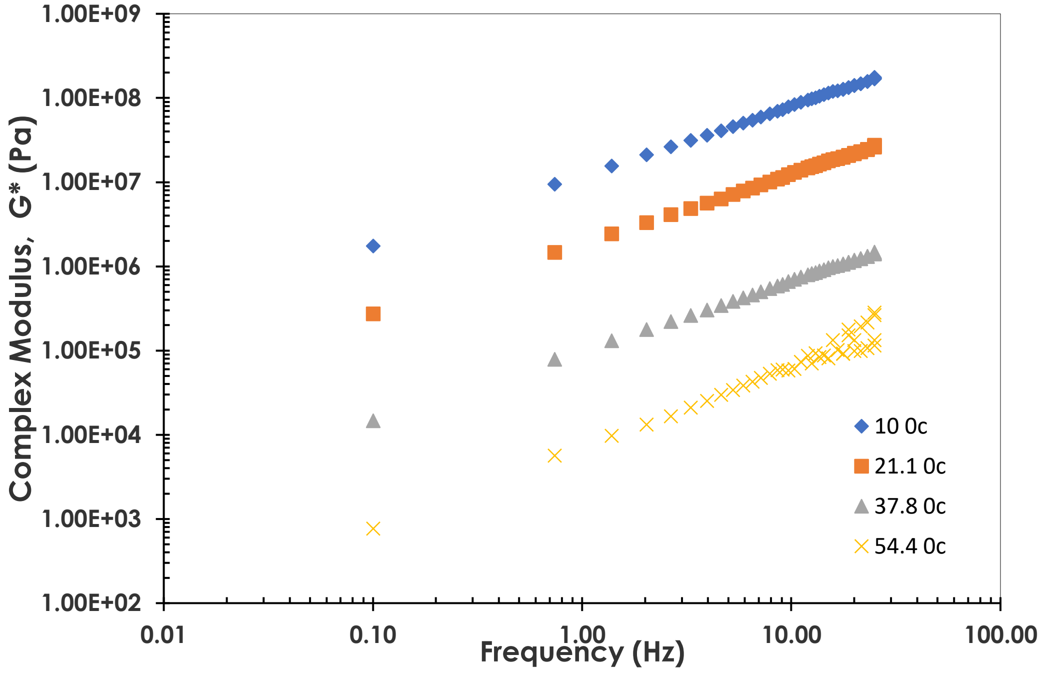


*Figure C-6. 6 % aged Binder*

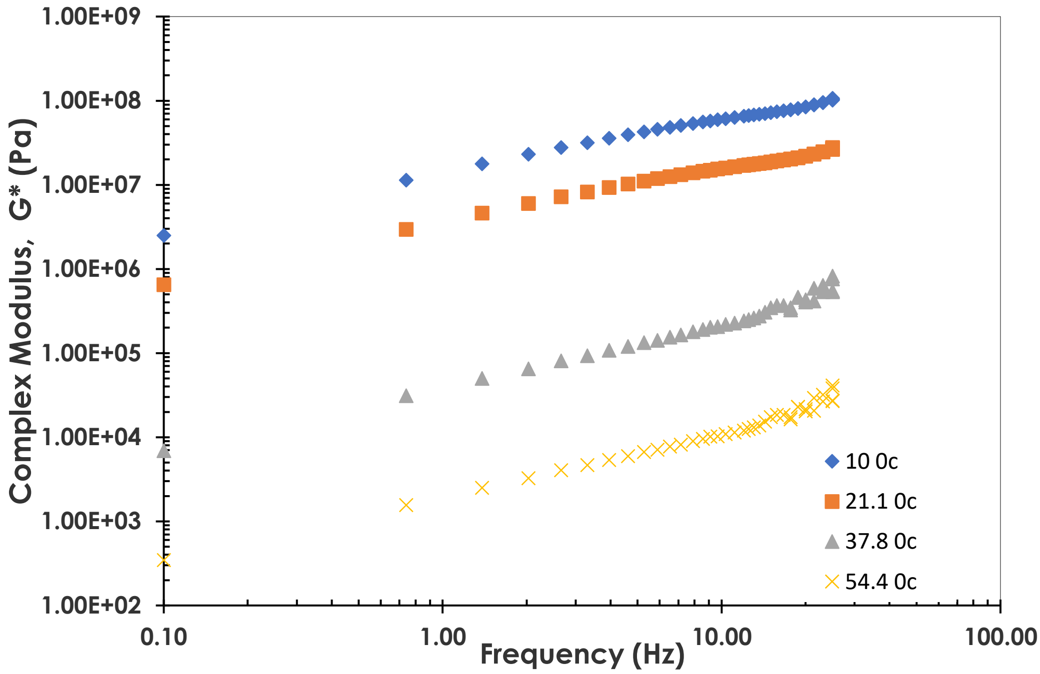


*Figure C-7. 6% Unaged Binder*

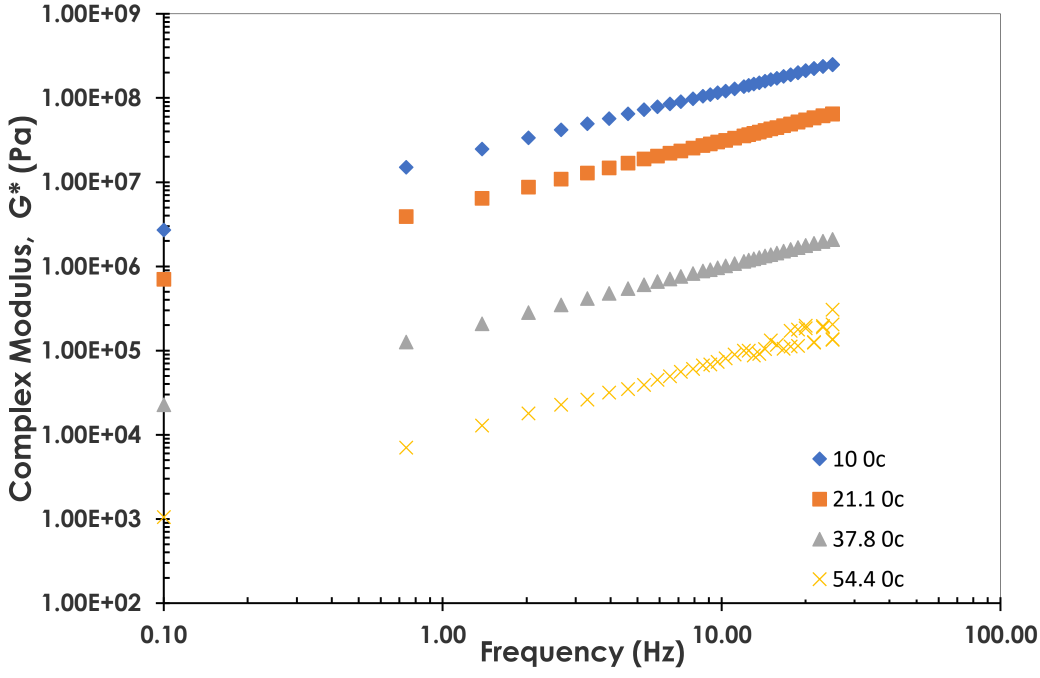


*Figure C-8. 9 % aged Binder*

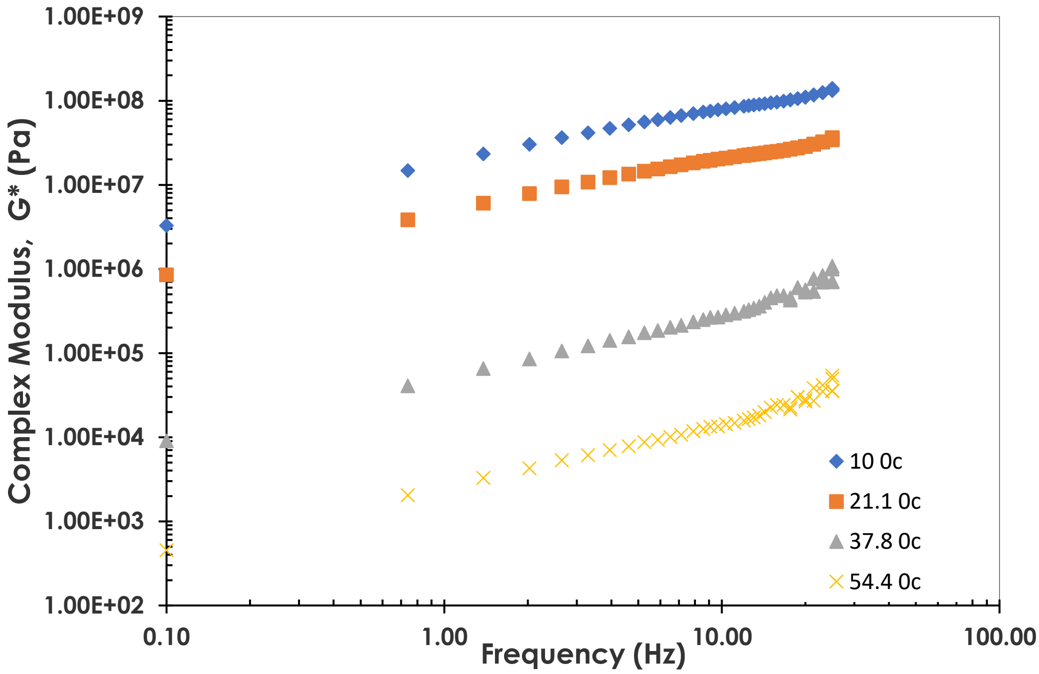


*Figure C-9. 12 % Unaged Binder*

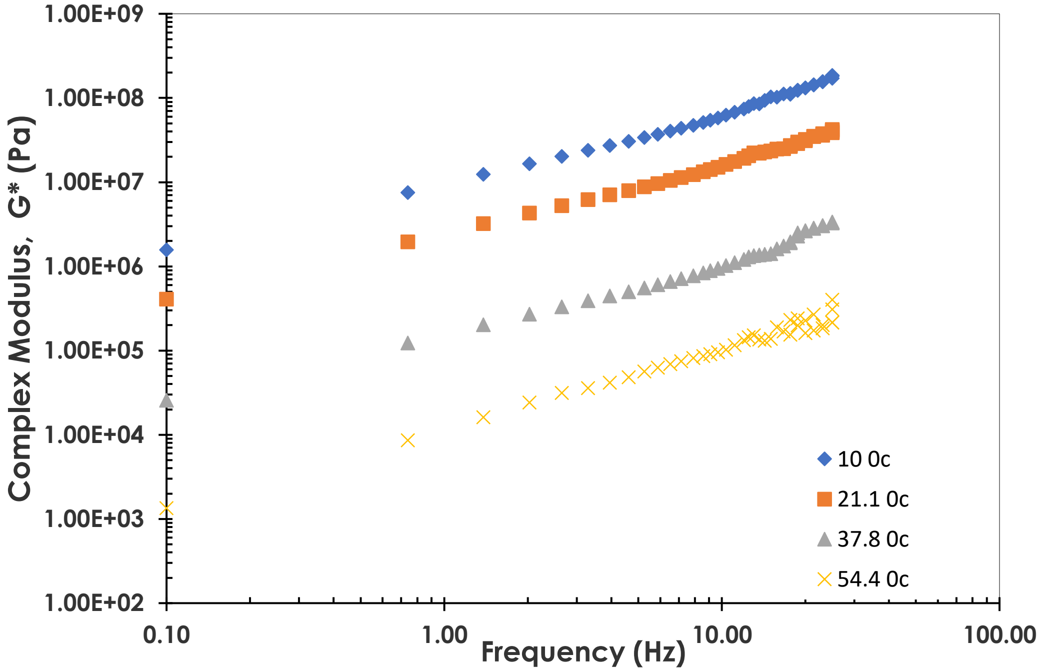


*Figure C-10. 12 % aged neat Binder*

# APPENDIX-D STATISTICAL ANALYSIS USING ANOVA

*Table 9. Statistical analysis for FST at f = 0.1 Hz using ANOVA*

Anova: Single Factor

SUMMARY

| *Groups* | *Count* | *Sum* | *Average* | *Variance* |
|---|---|---|---|---|
| 0 | 2 | 35335.608 | 17667.804 | 13848270 |
| 0.03 | 2 | 78063.471 | 39031.735 | 1.03E+08 |
| 0.06 | 2 | 131756.604 | 65878.302 | 2.61E+08 |
| 0.09 | 2 | 168572.357 | 84286.179 | 6.24E+08 |
| 0.12 | 2 | 186459.839 | 93229.919 | 7.86E+08 |

ANOVA

| *Source of Variation* | *SS* | *df* | *MS* | *F* | *P-value* | *F crit* |
|---|---|---|---|---|---|---|
| Between Groups | 7.921E+09 | 4 | 2.0E+09 | 5.53942 | 0.044211 | 11.39193 |
| Within Groups | 1.787E+09 | 5 | 3.6E+08 | | | |
| | | | | | | |
| Total | 9.708E+09 | 9 | | | | |

*Table 10. Statistical analysis for FST at f = 10 Hz using ANOVA*

Anova: Single Factor

SUMMARY

| *Groups* | *Count* | *Sum* | *Average* | *Variance* |
|---|---|---|---|---|
| 0 | 2 | 7152662.04 | 3576331.02 | 8.63E+11 |
| 0.03 | 2 | 28755724.25 | 14377862.12 | 1.92E+13 |
| 0.06 | 2 | 78200240.93 | 39100120.46 | 1.15E+14 |
| 0.09 | 2 | 85492096.04 | 42746048.02 | 1.7E+14 |
| 0.12 | 2 | 118145426.37 | 59072713.19 | 3.15E+14 |

ANOVA

| *Source of Variation* | *SS* | *df* | *MS* | *F* | *P-value* | *F crit* |
|---|---|---|---|---|---|---|
| Between Groups | 4.034E+15 | 4 | 1.00851E+15 | 8.131917 | 0.02052 | 11.39193 |
| Within Groups | 6.201E+14 | 5 | 1.24018E+14 | | | |
| Total | 4.654E+15 | 9 | | | | |

*Table 11. Statistical analysis for FST at f = 25 Hz using ANOVA*

Anova: Single Factor

SUMMARY

| *Groups* | *Count* | *Sum* | *Average* | *Variance* |
|---|---|---|---|---|
| 0 | 2 | 10852199.6 | 5426099.80 | 1.89E+11 |
| 0.03 | 2 | 48453632.95 | 24226816.47 | 5.71E+12 |
| 0.06 | 2 | 186780749.1 | 93390374.54 | 1.19E+14 |
| 0.09 | 2 | 180515678.8 | 90257839.41 | 7.93E+13 |
| 0.12 | 2 | 280055980.7 | 140027990.3 | 1.68E+14 |

ANOVA

| *Source of Variation* | *SS* | *df* | *MS* | *F* | *P-value* | *F crit* |
|---|---|---|---|---|---|---|
| Between Groups | 2.425E+16 | 4 | 6.06208E+15 | 81.48646 | 9.71E-05 | 11.39193 |
| Within Groups | 3.72E+14 | 5 | 7.43937E+13 | | | |
| Total | 2.462E+16 | 9 | | | | |

*Table 12.* Performance Grade Determination for neat Unaged asphalt binder

| Time (s) | Temperature (°C) | Frequency (Hz) | Phase Angle (°) | Complex Modulus (Pa) | Elastic Modulus (Pa) | Viscous Modulus (Pa) | Complex Viscosity (Pas) | Shear Stress (Pa) | Strain (% ) |
|---|---|---|---|---|---|---|---|---|---|
| 40.36 | 52.05 | 1.60 | 83.21 | 13636.00 | 1611.70 | 13540.00 | 1360.00 | 1655.90 | 0.12 |
| 40.39 | 58.02 | 1.60 | 85.34 | 5357.40 | 434.94 | 5339.70 | 534.33 | 639.36 | 0.12 |
| 40.38 | 64.00 | 1.60 | 86.66 | 2282.00 | 132.97 | 2278.10 | 227.60 | 274.72 | 0.12 |
| 40.37 | 70.00 | 1.60 | 87.66 | 1007.40 | 41.21 | 1006.50 | 100.47 | 120.31 | 0.12 |
| 40.99 | 75.89 | 1.60 | 87.92 | 477.14 | 17.28 | 476.83 | 47.59 | 57.08 | 0.12 |

*Table 13* Performance Grade Determination for 3 % Unaged asphalt binder

| Time (s) | Temperature (°C) | Frequency (Hz) | Phase Angle (°) | Complex Modulus (Pa) | Elastic Modulus (Pa) | Viscous Modulus (Pa) | Complex Viscosity (Pas) | Shear Stress (Pa) | Strain ( %) |
|---|---|---|---|---|---|---|---|---|---|
| 40.364 | 57.99 | 1.60E+00 | 86.55 | 3.68E+03 | 2.22E+02 | 3.67E+03 | 3.67E+02 | 4.49E+02 | 0.12 |
| 40.349 | 63.92 | 1.60E+00 | 87.4 | 1.83E+03 | 8.31E+01 | 1.83E+03 | 1.83E+02 | 2.28E+02 | 0.12 |
| 40.363 | 69.9 | 1.60E+00 | 88.14 | 8.65E+02 | 2.81E+01 | 8.64E+02 | 8.62E+01 | 1.06E+02 | 0.12 |

*Table 14 Performance Grade Determination for 6 % Unaged asphalt binder*

| Time (s) | Temperature (°C) | Frequency (Hz) | Phase Angle (°) | Complex Modulus (Pa) | Elastic Modulus (Pa) | Viscous Modulus (Pa) | Complex Viscosity (Pas) | Shear Stress (Pa) | Strain (% ) |
|---|---|---|---|---|---|---|---|---|---|
| 40.382 | 58 | 1.60E+00 | 86.55 | 4.02E+03 | 2.42E+02 | 4.01E+03 | 4.01E+02 | 4.86E+02 | 0.12 |
| 40.321 | 63.92 | 1.60E+00 | 87.57 | 1.88E+03 | 8.00E+01 | 1.88E+03 | 1.88E+02 | 2.27E+02 | 0.12 |
| 40.32 | 70.02 | 1.60E+00 | 88.33 | 8.24E+02 | 2.41E+01 | 8.23E+02 | 8.22E+01 | 9.86E+01 | 0.12 |

*Table 15. Performance Grade Determination for 9 % Unaged asphalt binder*

| Time (s) | Temperature (°C) | Frequency (Hz) | Phase Angle (°) | Complex Modulus (Pa) | Elastic Modulus (Pa) | Viscous Modulus (Pa) | Complex Viscosity (Pas) | Shear Stress (Pa) | Strain ( %) |
|---|---|---|---|---|---|---|---|---|---|
| 40.326 | 58.01 | 1.60E+00 | 86.55 | 3.85E+03 | 2.31E+02 | 3.84E+03 | 3.84E+02 | 4.61E+02 | 0.12 |
| 40.331 | 64.06 | 1.60E+00 | 87.73 | 1.67E+03 | 6.62E+01 | 1.67E+03 | 1.67E+02 | 1.99E+02 | 0.12 |
| 40.362 | 69.88 | 1.60E+00 | 88.51 | 8.08E+02 | 2.10E+01 | 8.07E+02 | 8.05E+01 | 9.52E+01 | 0.12 |

*Table 16.* Performance Grade Determination for 12 % Unaged asphalt binder

| Time | Temperature | Frequency | Phase Angle | Complex Modulus | Elastic Modulus | Viscous Modulus | Complex Viscosity | Shear Stress | Strain |
|---|---|---|---|---|---|---|---|---|---|
| (s) | (°C) | (Hz) | (°) | (Pa) | (Pa) | (Pa) | (Pas) | (Pa) | ( %) |
| 39.779 | 58 | 1.60E+00 | 86.76 | 4.06E+03 | 2.30E+02 | 4.06E+03 | 4.05E+02 | 4.84E+02 | 0.12 |
| 40.369 | 63.9 | 1.60E+00 | 87.8 | 1.98E+03 | 7.61E+01 | 1.98E+03 | 1.98E+02 | 2.36E+02 | 0.12 |
| 40.435 | 70.02 | 1.60E+00 | 88.5 | 8.34E+02 | 2.18E+01 | 8.34E+02 | 8.32E+01 | 1.00E+02 | 0.12 |

*Table 17.* Performance Grade Determination for neat RTFO Aged asphalt binder

| Time | Temperature | Frequency | Phase Angle | Complex Modulus | Elastic Modulus | Viscous Modulus | Complex Viscosity | Shear Stress | Strain |
|---|---|---|---|---|---|---|---|---|---|
| (s) | (°C) | (Hz) | (°) | (Pa) | (Pa) | (Pa) | (Pas) | (Pa) | ( %) |
| 40.358 | 64.04 | 1.60E+00 | 85.83 | 2.66E+03 | 1.94E+02 | 2.66E+03 | 2.66E+02 | 2.67E+02 | 0.10 |
| 40.383 | 69.95 | 1.60E+00 | 87.03 | 1.21E+03 | 6.24E+01 | 1.20E+03 | 1.20E+02 | 1.20E+02 | 0.10 |

*Table 18.* Performance Grade Determination for 3 % RTFO Aged asphalt binder

| Time | Temperature | Frequency | Phase Angle | Complex Modulus | Elastic Modulus | Viscous Modulus | Complex Viscosity | Shear Stress | Strain |
|---|---|---|---|---|---|---|---|---|---|
| (s) | (°C) | (Hz) | (°) | (Pa) | (Pa) | (Pa) | (Pas) | (Pa) | (% ) |
| 40.302 | 51.97 | 1.60E+00 | 82.01 | 1.51E+04 | 2.09E+03 | 1.49E+04 | 1.50E+03 | 1.81E+03 | 0.12 |
| 40.303 | 58.01 | 1.60E+00 | 84.34 | 5.98E+03 | 5.90E+02 | 5.95E+03 | 5.97E+02 | 7.26E+02 | 0.12 |
| 40.265 | 64 | 1.60E+00 | 86.12 | 2.78E+03 | 1.88E+02 | 2.77E+03 | 2.77E+02 | 3.40E+02 | 0.12 |
| 40.264 | 69.98 | 1.60E+00 | 86.94 | 1.12E+03 | 5.99E+01 | 1.12E+03 | 1.12E+02 | 1.35E+02 | 0.12 |

*Table 19.* Performance Grade Determination for 6 % RTFO Aged asphalt binder

| Time | Temperature | Frequency | Phase Angle | Complex Modulus | Elastic Modulus | Viscous Modulus | Complex Viscosity | Shear Stress | Strain |
|---|---|---|---|---|---|---|---|---|---|
| (s) | (°C) | (Hz) | (°) | (Pa) | (Pa) | (Pa) | (Pas) | (Pa) | ( %) |
| 40.378 | 52.03 | 1.60E+00 | 81.59 | 1.88E+04 | 2.75E+03 | 1.86E+04 | 1.88E+03 | 2.28E+03 | 0.12 |
| 40.425 | 57.93 | 1.60E+00 | 83.99 | 7.78E+03 | 8.15E+02 | 7.74E+03 | 7.76E+02 | 9.32E+02 | 0.12 |
| 40.371 | 64 | 1.60E+00 | 85.92 | 3.17E+03 | 2.26E+02 | 3.16E+03 | 3.16E+02 | 3.76E+02 | 0.12 |
| 40.43 | 69.92 | 1.60E+00 | 87.13 | 1.48E+03 | 7.40E+01 | 1.48E+03 | 1.47E+02 | 1.78E+02 | 0.12 |

*Table 20.* Performance Grade Determination for 9 % RTFO Aged asphalt binder

| Time | Temperature | Frequency | Phase Angle | Complex Modulus | Elastic Modulus | Viscous Modulus | Complex Viscosity | Shear Stress | Strain |
|---|---|---|---|---|---|---|---|---|---|
| (s) | (°C) | (Hz) | (°) | (Pa) | (Pa) | (Pa) | (Pas) | (Pa) | ( %) |
| 40.365 | 51.98 | 1.60E+00 | 80.8 | 2.17E+04 | 3.47E+03 | 2.14E+04 | 2.16E+03 | 2.62E+03 | 0.12 |
| 40.433 | 57.99 | 1.60E+00 | 83.44 | 8.49E+03 | 9.70E+02 | 8.44E+03 | 8.47E+02 | 1.01E+03 | 0.12 |
| 40.361 | 63.95 | 1.60E+00 | 85.3 | 3.60E+03 | 2.95E+02 | 3.59E+03 | 3.59E+02 | 4.32E+02 | 0.12 |
| 40.436 | 70.06 | 1.60E+00 | 86.63 | 1.53E+03 | 8.96E+01 | 1.52E+03 | 1.52E+02 | 1.83E+02 | 0.12 |

*Table 21. Performance Grade Determination for 12 % RTFO Aged asphalt binder*

| Time | Temperature | Frequency | Phase Angle | Complex Modulus | Elastic Modulus | Viscous Modulus | Complex Viscosity | Shear Stress | Strain |
|---|---|---|---|---|---|---|---|---|---|
| (s) | (°C) | (Hz) | (°) | (Pa) | (Pa) | (Pa) | (Pas) | (Pa) | ( %) |
| 40.424 | 51.98 | 1.60E+00 | 81.36 | 1.76E+04 | 2.64E+03 | 1.74E+04 | 1.76E+03 | 2.14E+03 | 0.12 |
| 40.435 | 58.04 | 1.60E+00 | 83.66 | 7.50E+03 | 8.28E+02 | 7.45E+03 | 7.48E+02 | 9.10E+02 | 0.12 |
| 40.361 | 63.95 | 1.60E+00 | 85.38 | 3.38E+03 | 2.72E+02 | 3.37E+03 | 3.37E+02 | 4.08E+02 | 0.12 |
| 40.361 | 69.99 | 1.60E+00 | 86.85 | 1.58E+03 | 8.69E+01 | 1.58E+03 | 1.58E+02 | 1.93E+02 | 0.12 |

# APPENDIX E - MULTI STRESS CREEP AND RECOVERY TEST RESULTS

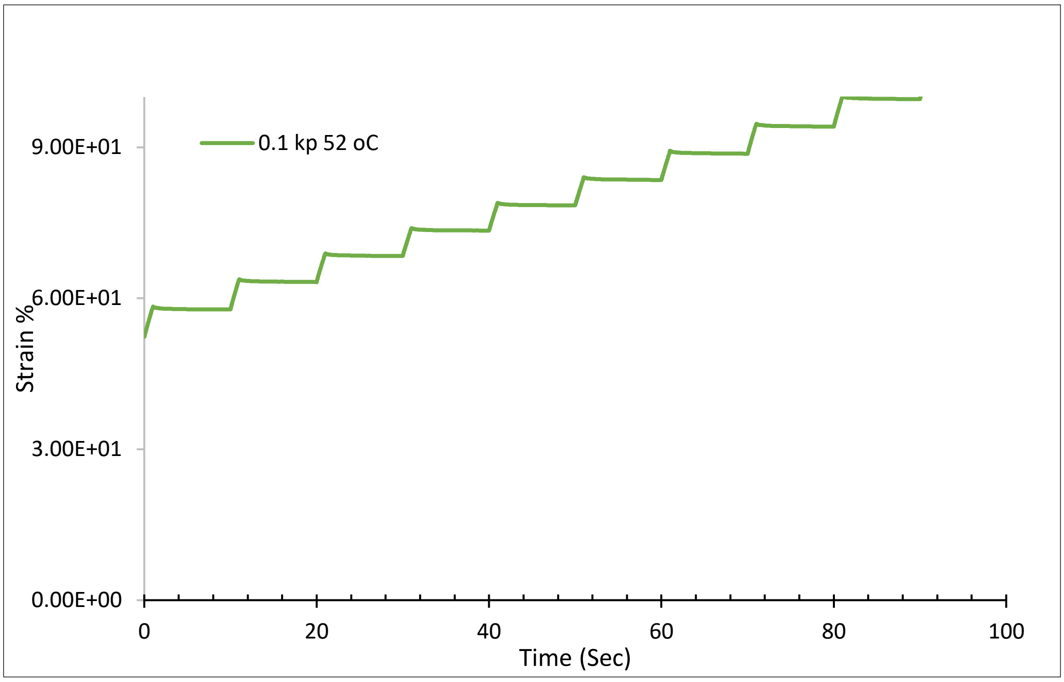


*Figure E-1. 0 % ANSS at 100 Pa for 52 $^{0}$c*

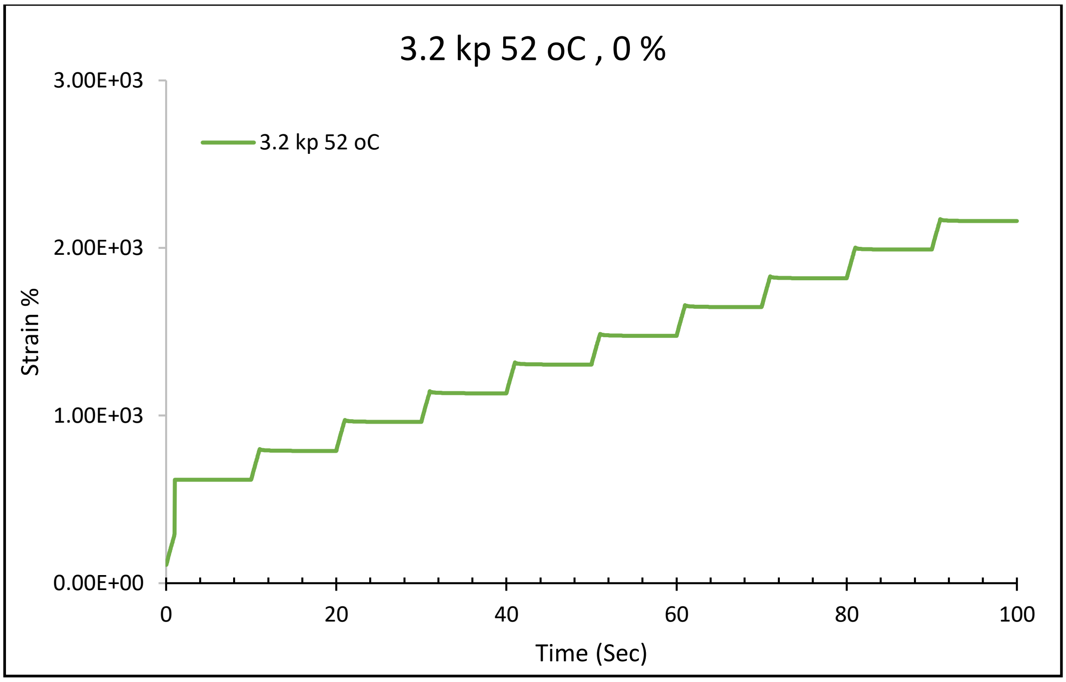


*Figure E-2. 0 % ANSS at 3200 Pa for 52 $^{0}$c*

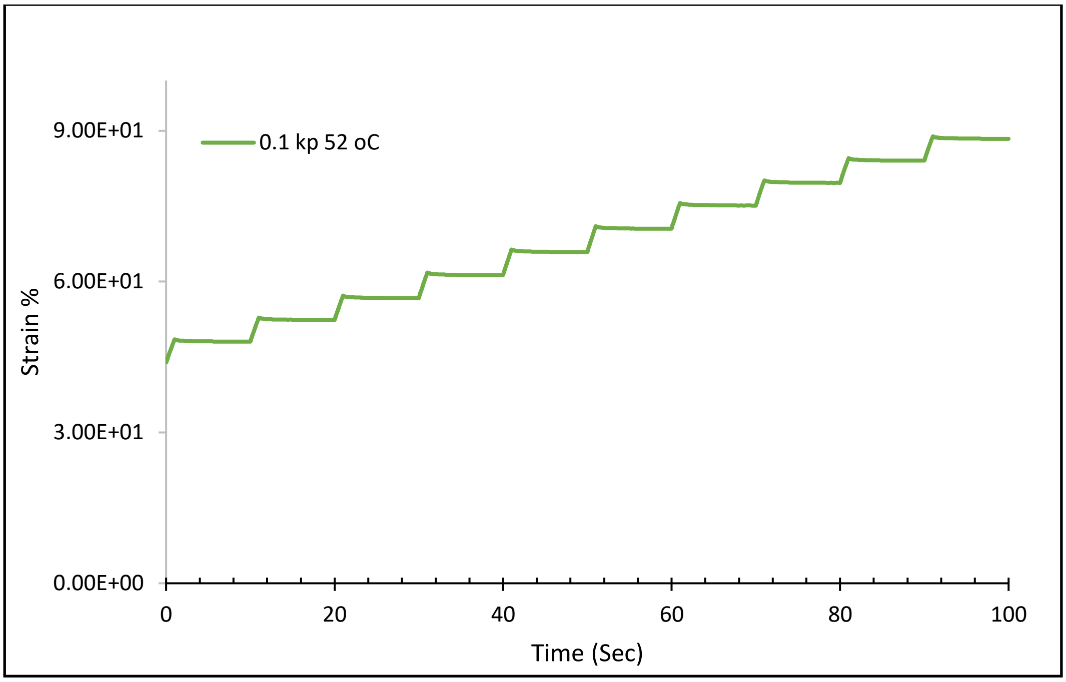


*Figure E-3. 3 % ANSS at 100 Pa for 52 $^{0}$c*

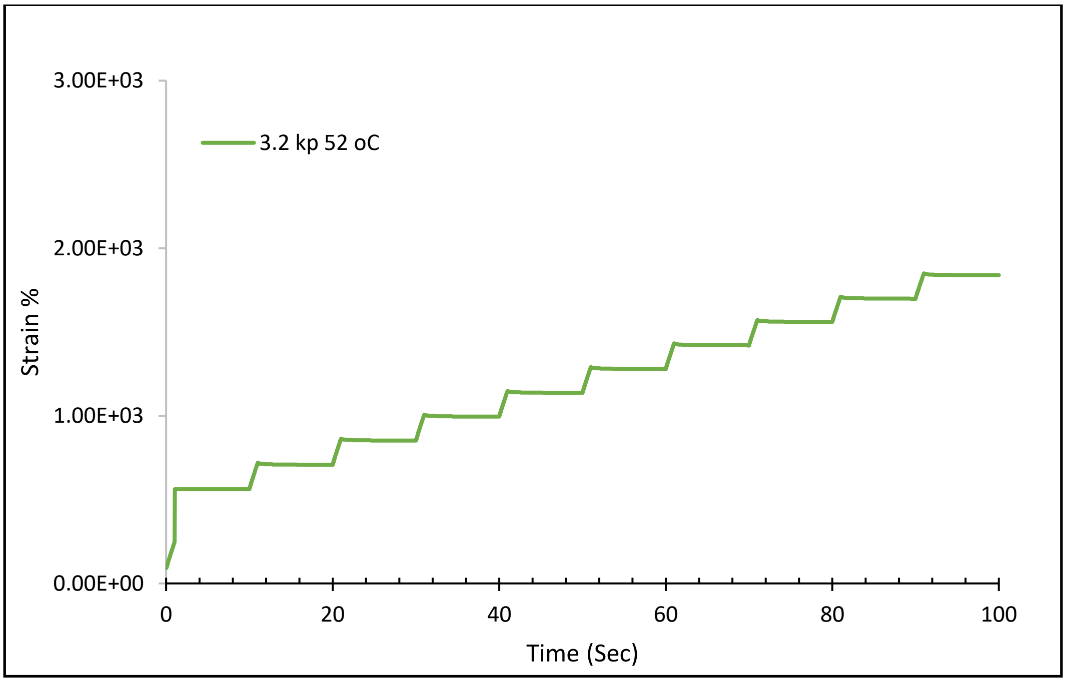


*Figure E-4. 3 % ANSS at 3200 Pa for 52 $^{0}$c*

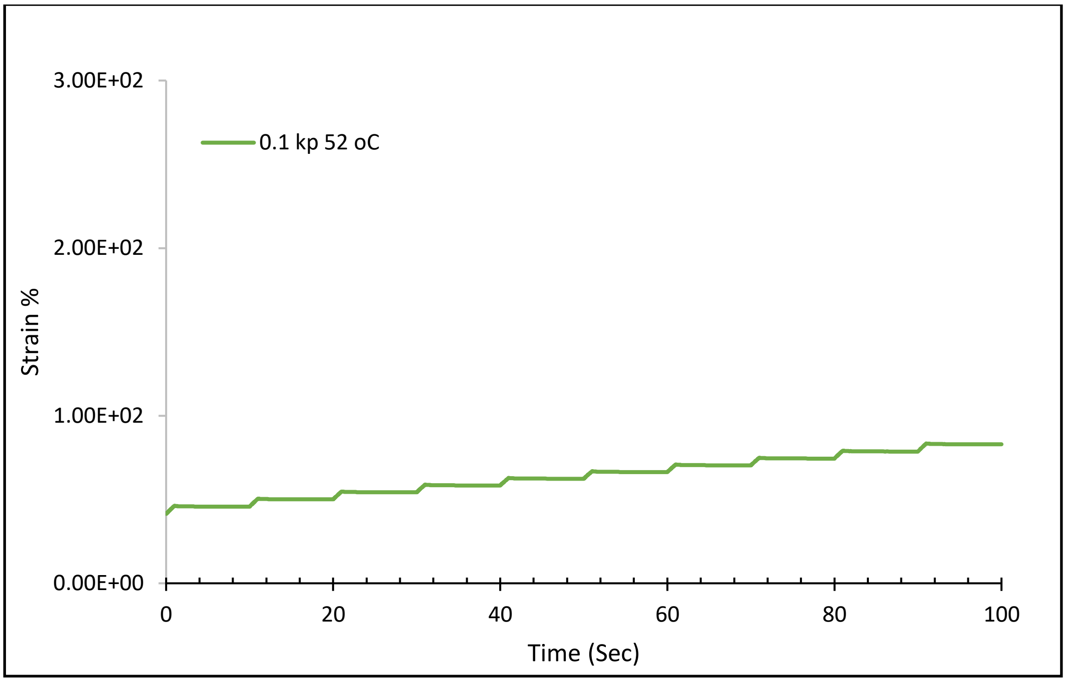


*Figure E-5. 6 % ANSS at 100 Pa for 52 $^0$c*

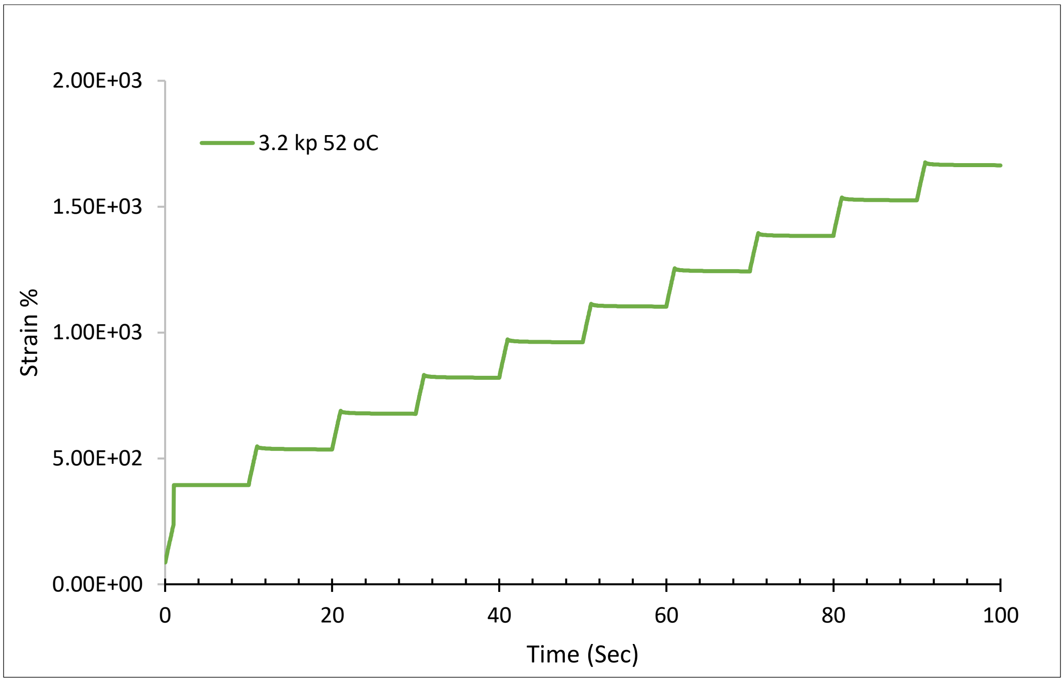


*Figure E-6. 6 % ANSS at 3200 Pa for 52 $^0$c*

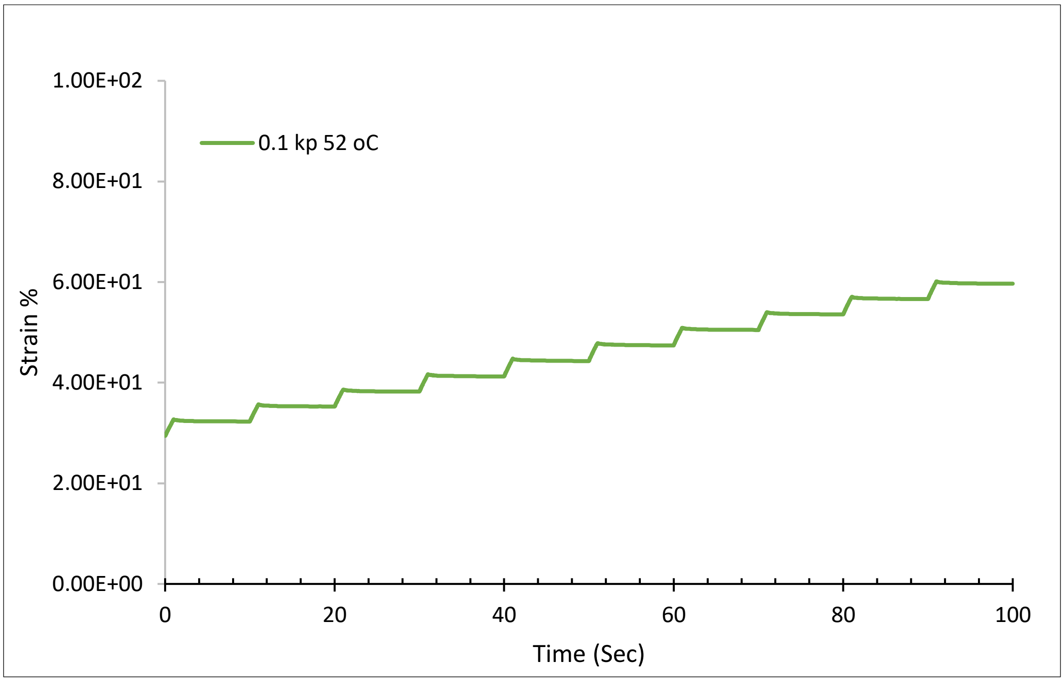


*Figure E-7. 9 % ANSS at 100 Pa for 52 $^{0}$c*

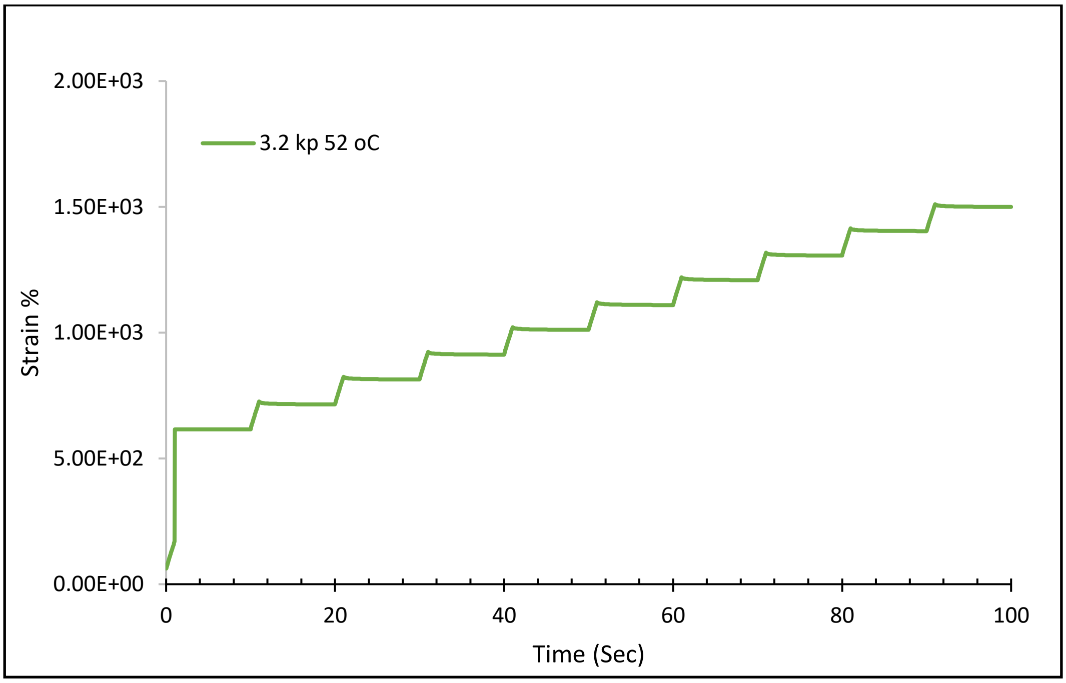


*Figure E-8. 9 % ANSS at 3200 Pa for 52 $^{0}$c*

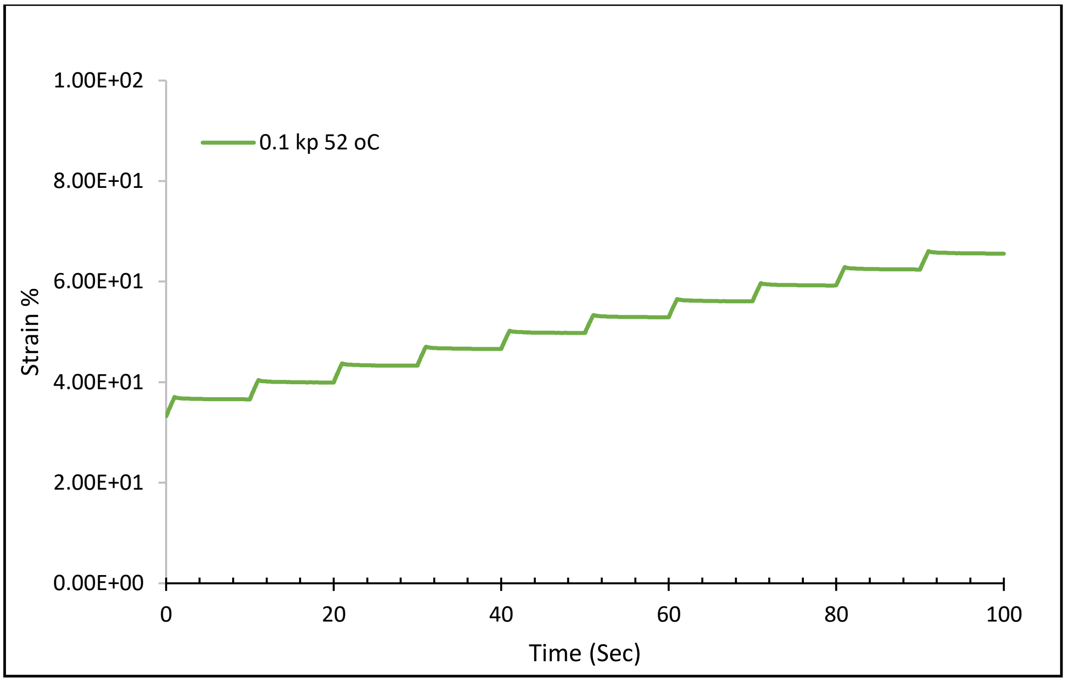


*Figure E-9. 12 % ANSS at 100 Pa for 52 $^0$c*

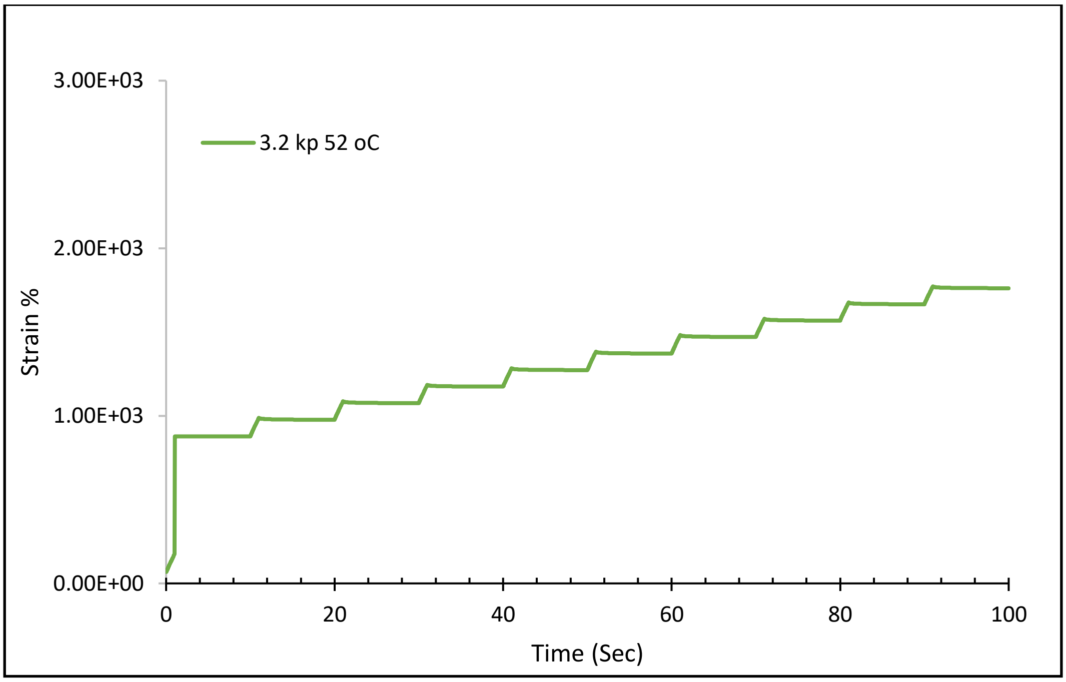


*Figure E-10. 12 % ANSS at 3200 Pa for 52 $^0$c*

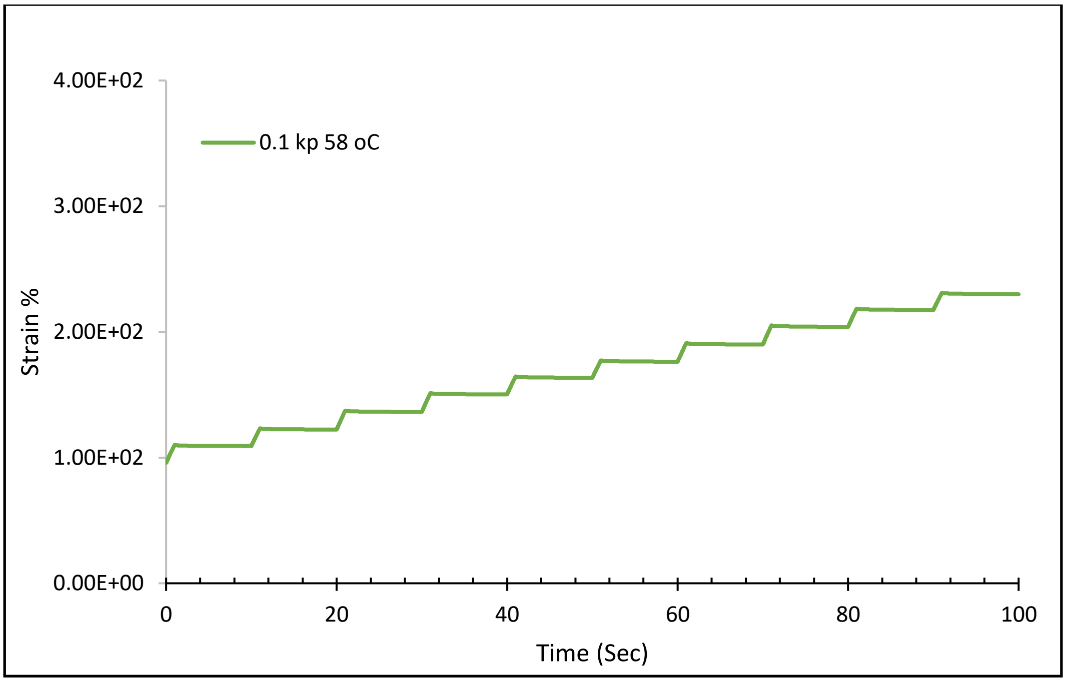


*Figure E-11. 0 % ANSS at 100 Pa for 58 $^0$c*

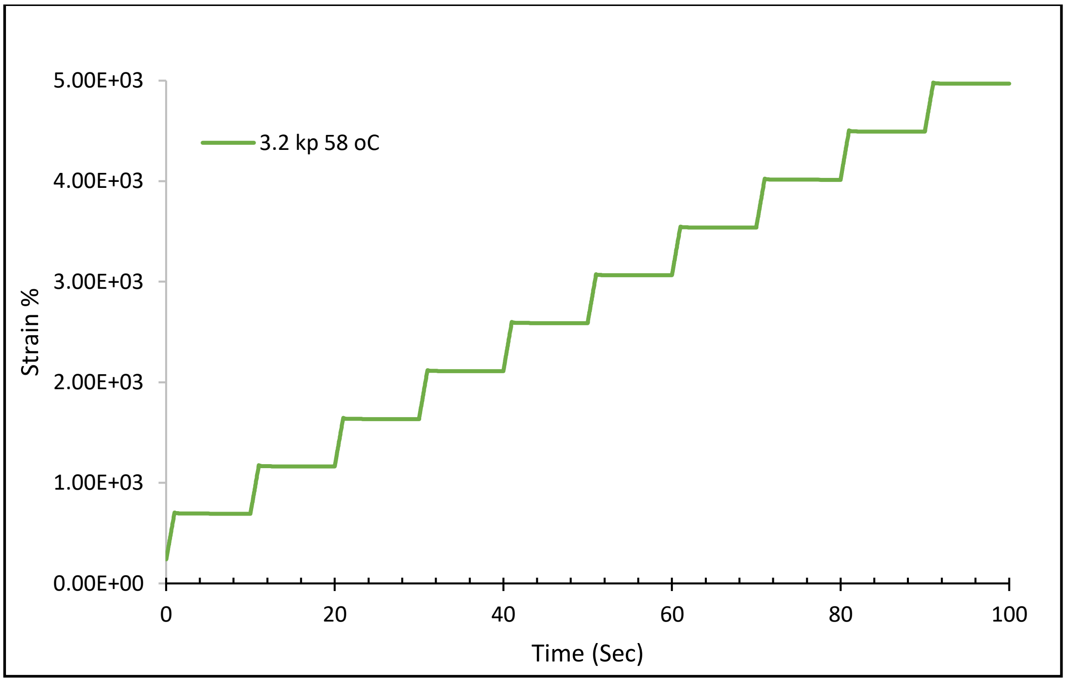


*Figure E-12. 0 % ANSS at 3200 Pa for 58 $^0$c*

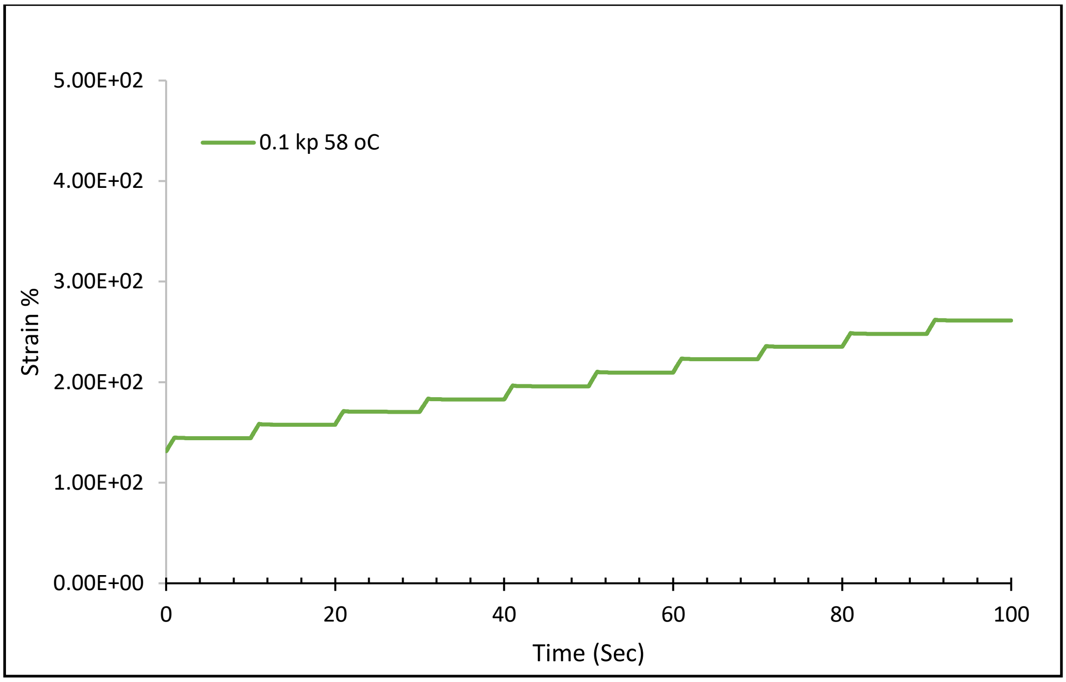


*Figure E-13. 3 % ANSS at 100 Pa for 58 $^0$c*

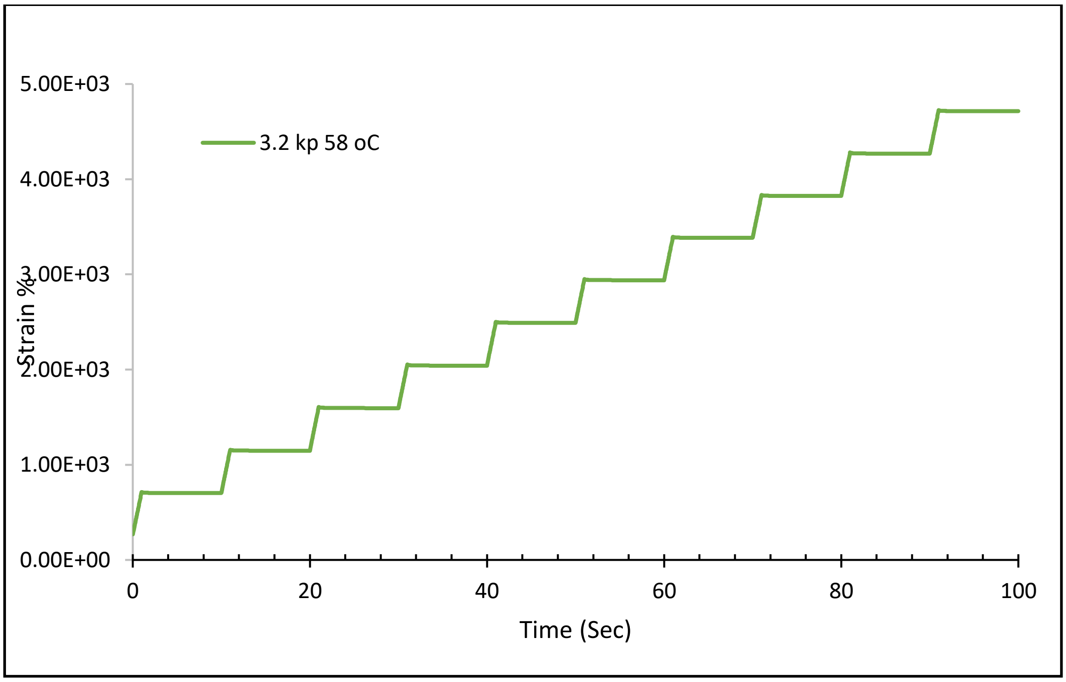


*Figure E-14. 3 % ANSS at 3200 Pa for 58 $^0$c*

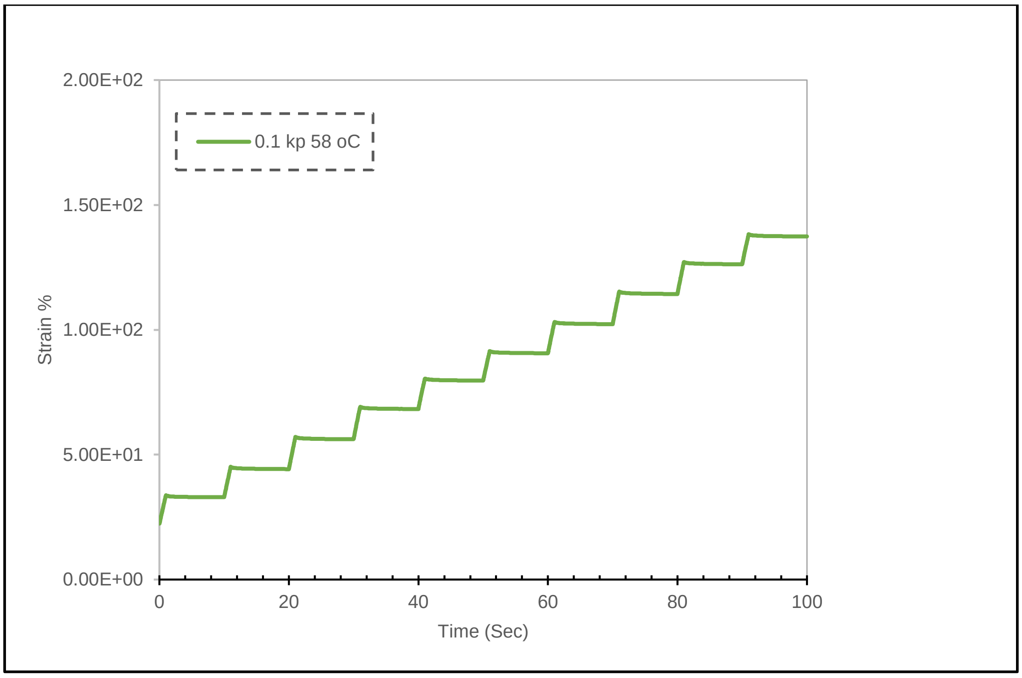


*Figure E-15. 6 % ANSS at 100 Pa for 58 $^0$c*

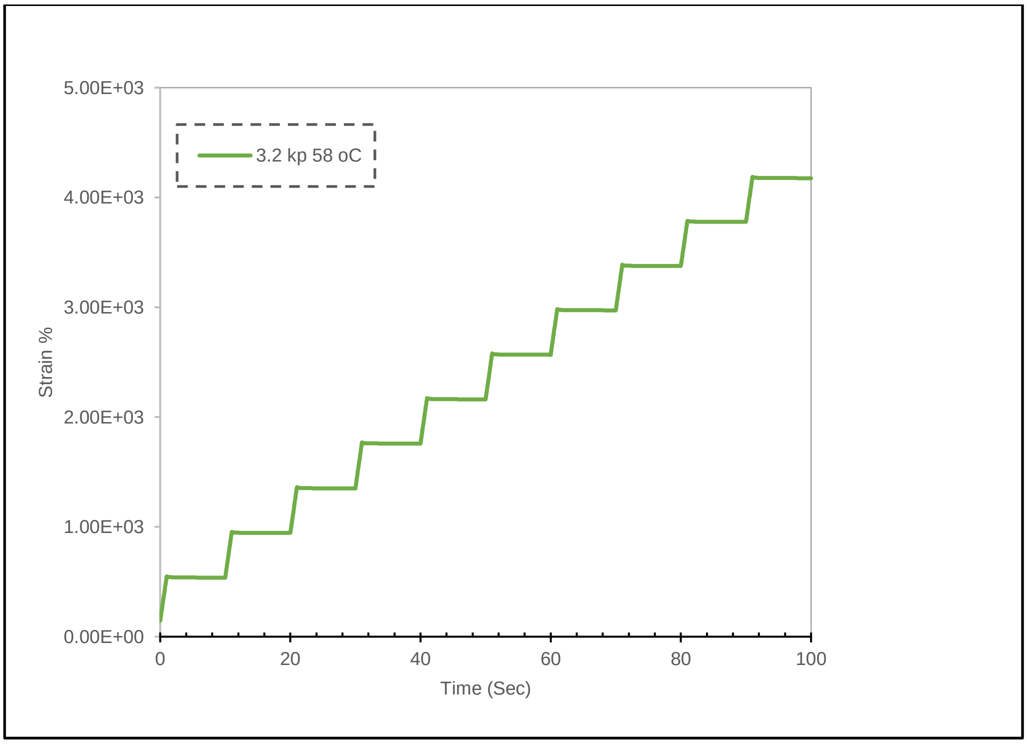


*Figure E-16. 6 % ANSS at 3200 Pa for 58 $^0$c*

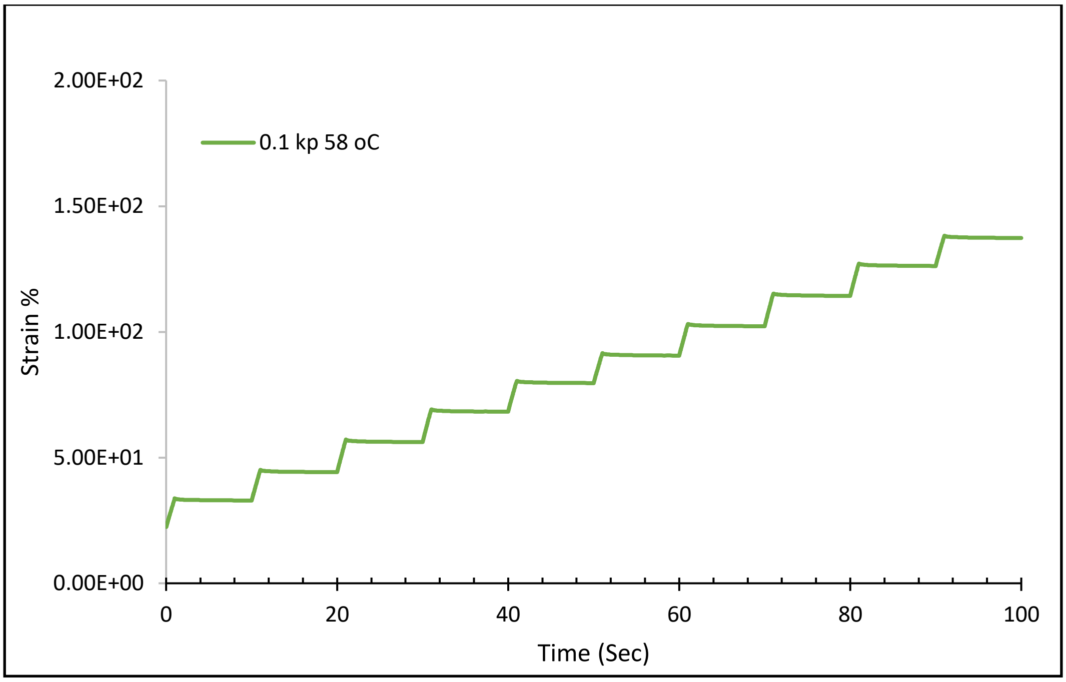


*Figure E-17. 9 % ANSS at 100 Pa for 58 $^0$c*

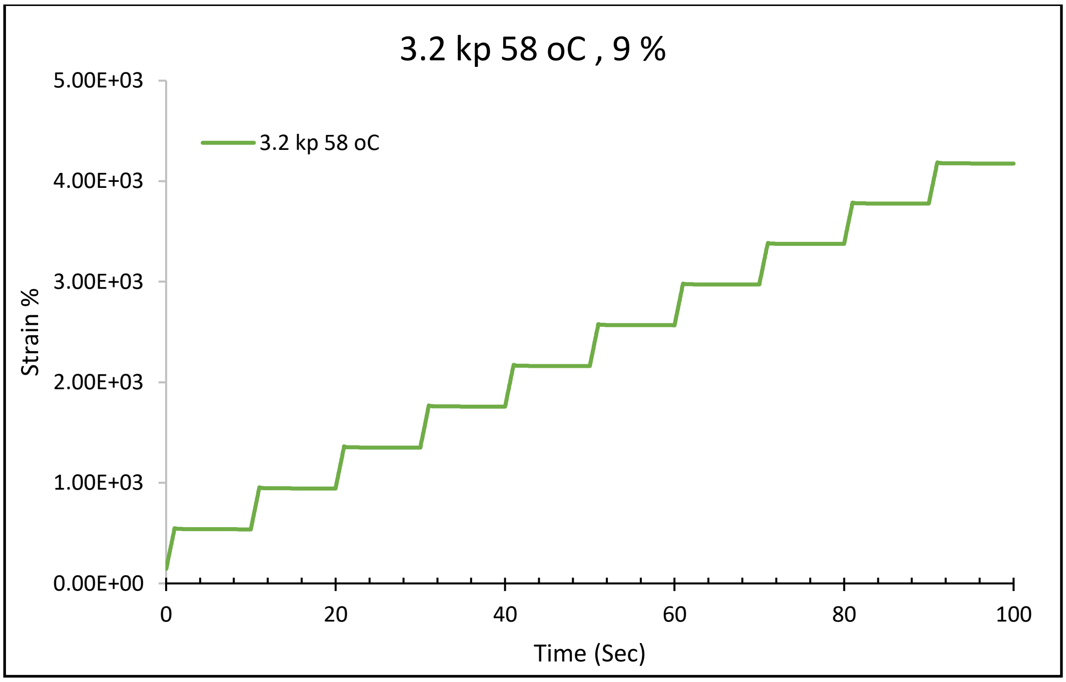


*Figure E-18. 9 % ANSS at 3200 Pa for 58 $^0$c*

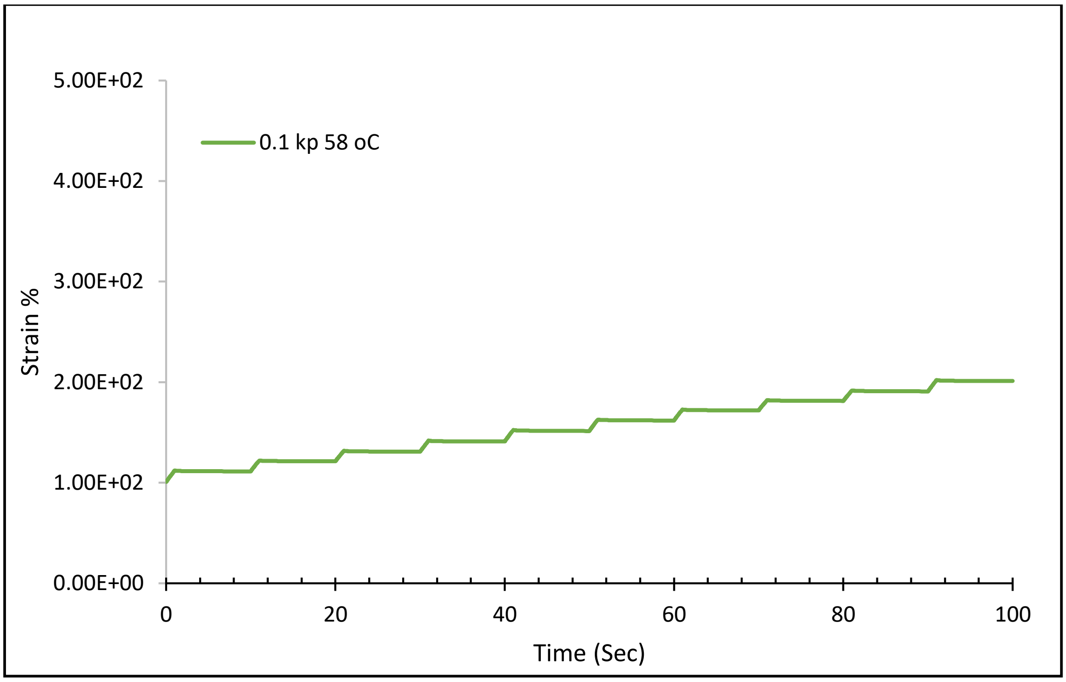


*Figure E-19. 12 % ANSS at 100 Pa for 58 $^{0}$c*

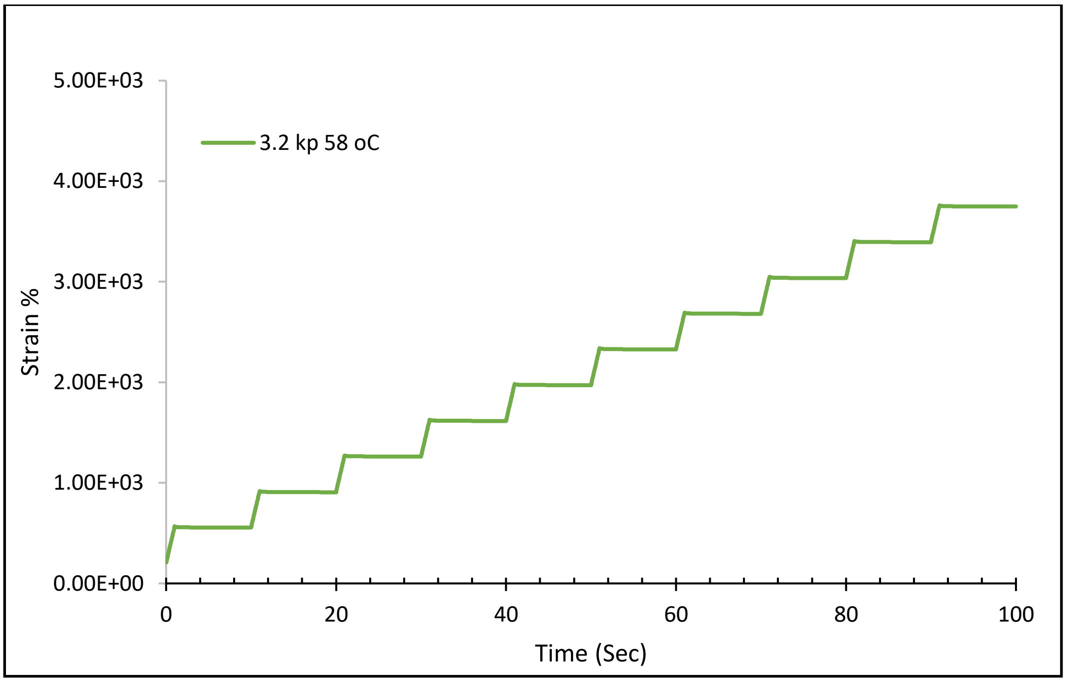


*Figure E-20. 12 % ANSS at 3200 Pa for 58 $^{0}$c*

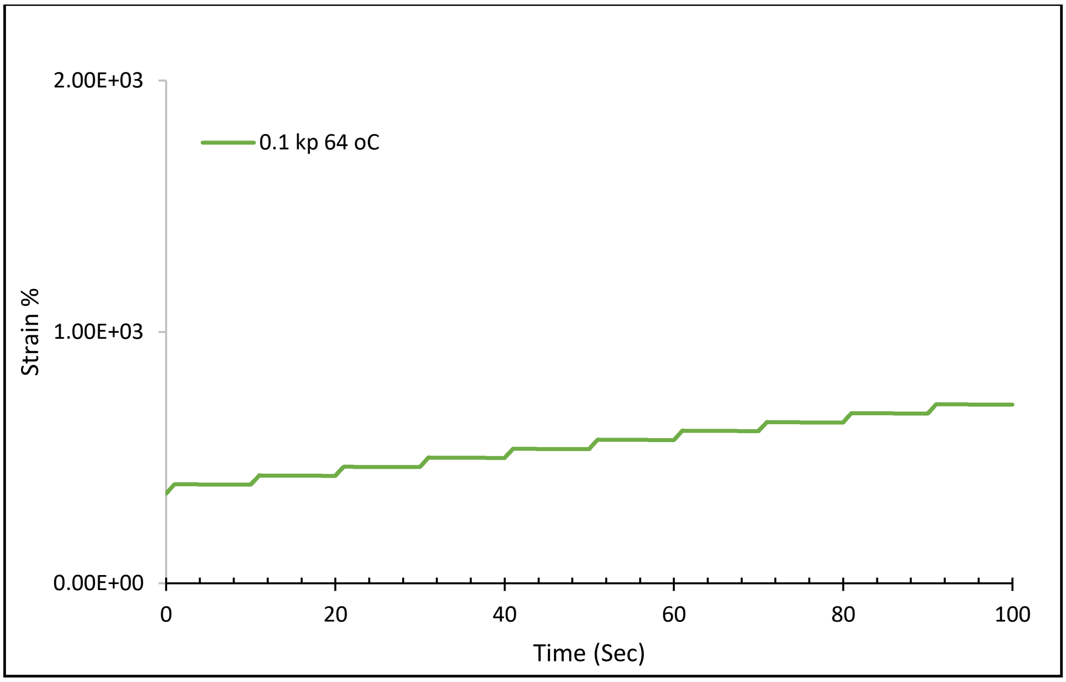


*Figure E-21. 0 % ANSS at 100 Pa for 64 $^{0}$c*

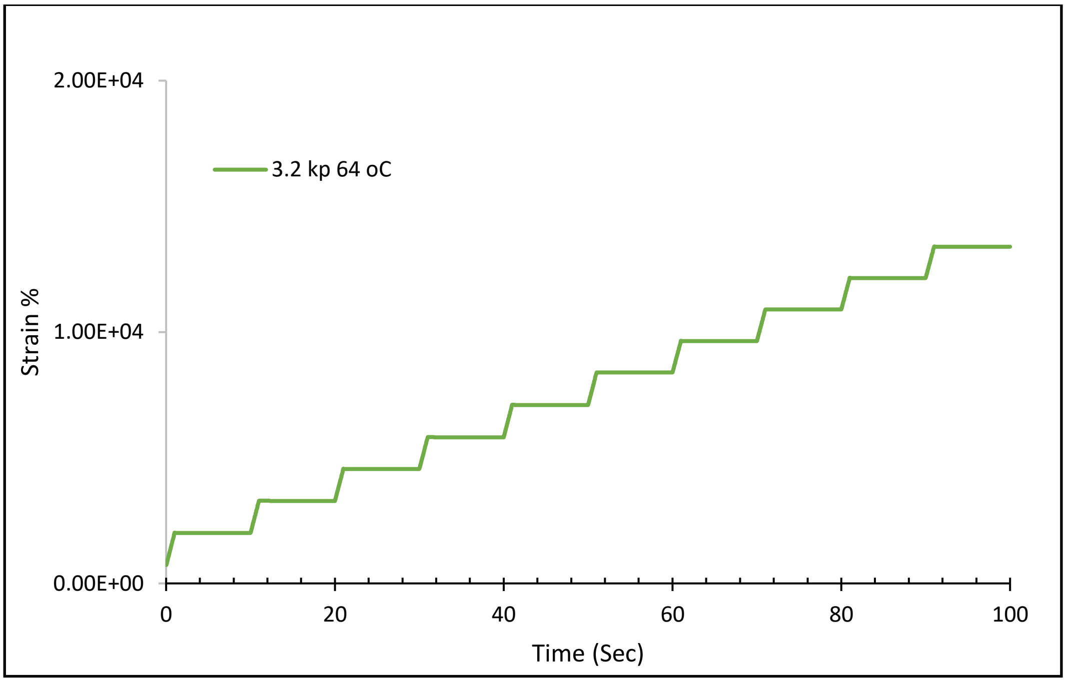


*Figure E-22. 0 % ANSS at 3200 Pa for 64 $^{0}$c*

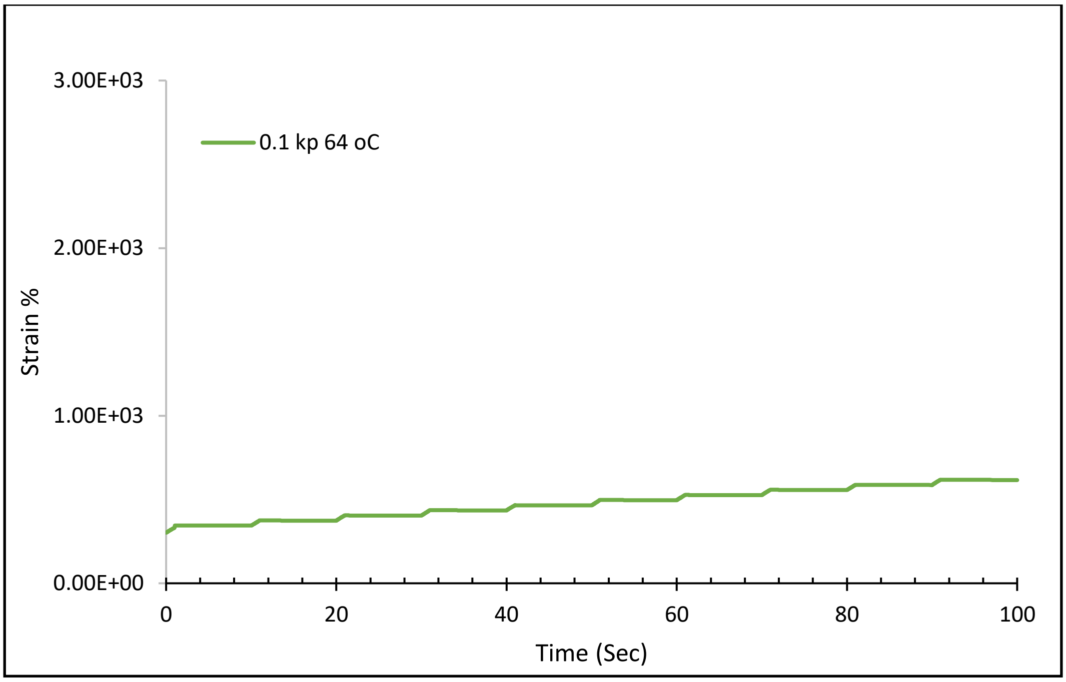


*Figure E-23. 3 % ANSS at 100 Pa for 64 $^{0}$c*

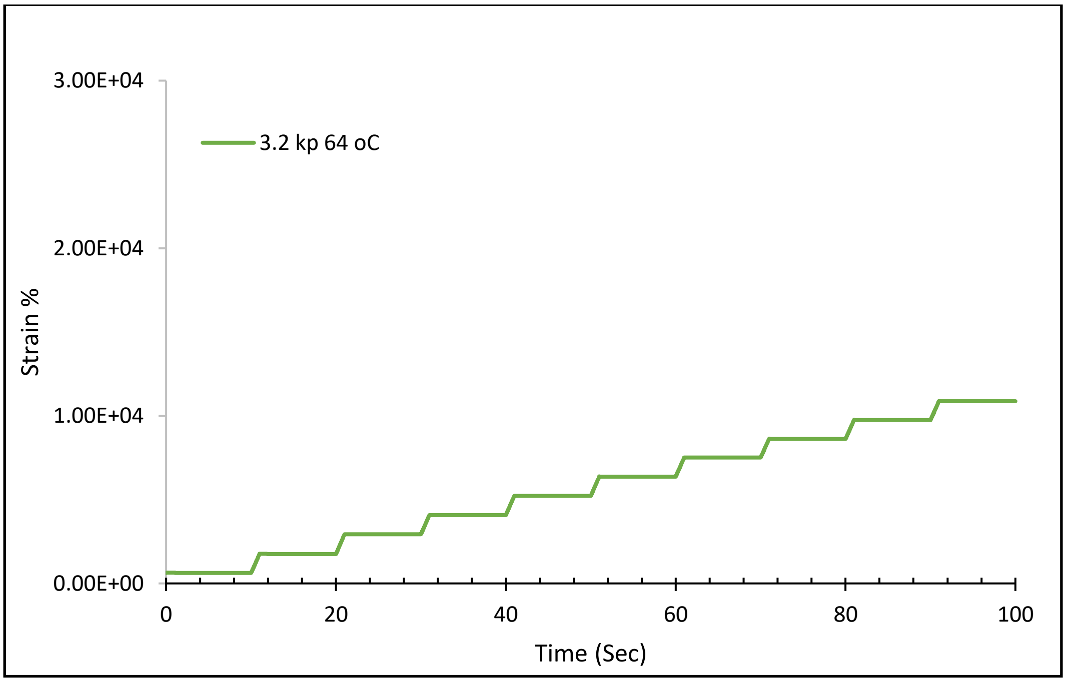


*Figure E-24. 3 % ANSS at 3200 Pa for 64 $^{0}$c*

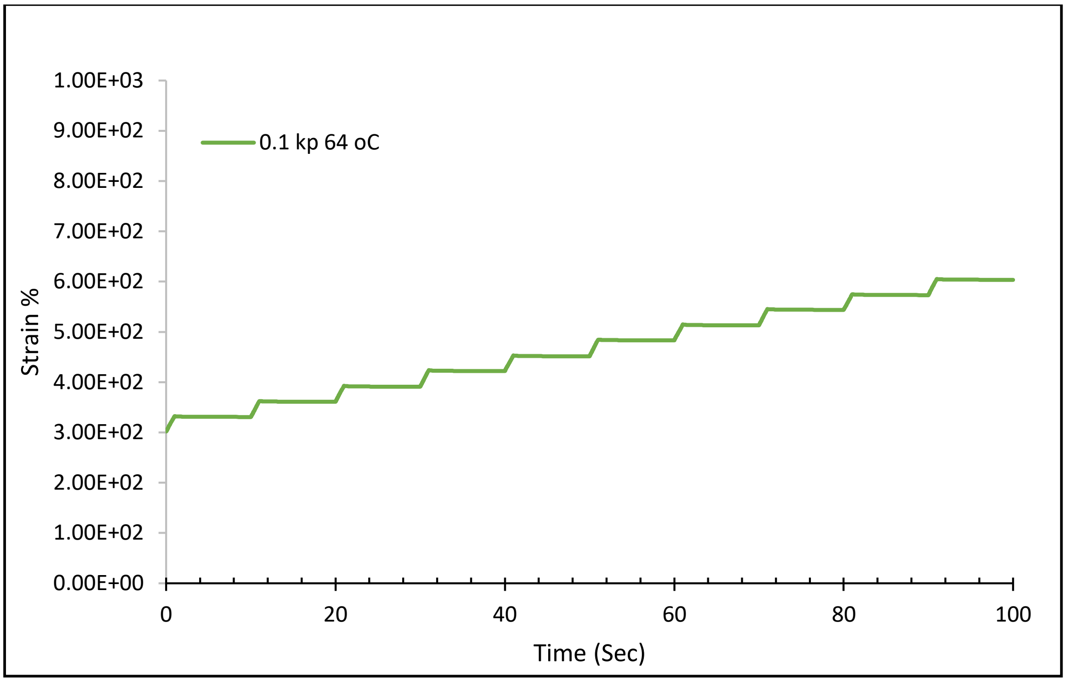


*Figure E-25. 6 % ANSS at 100 Pa for 64 $^{0}$c*

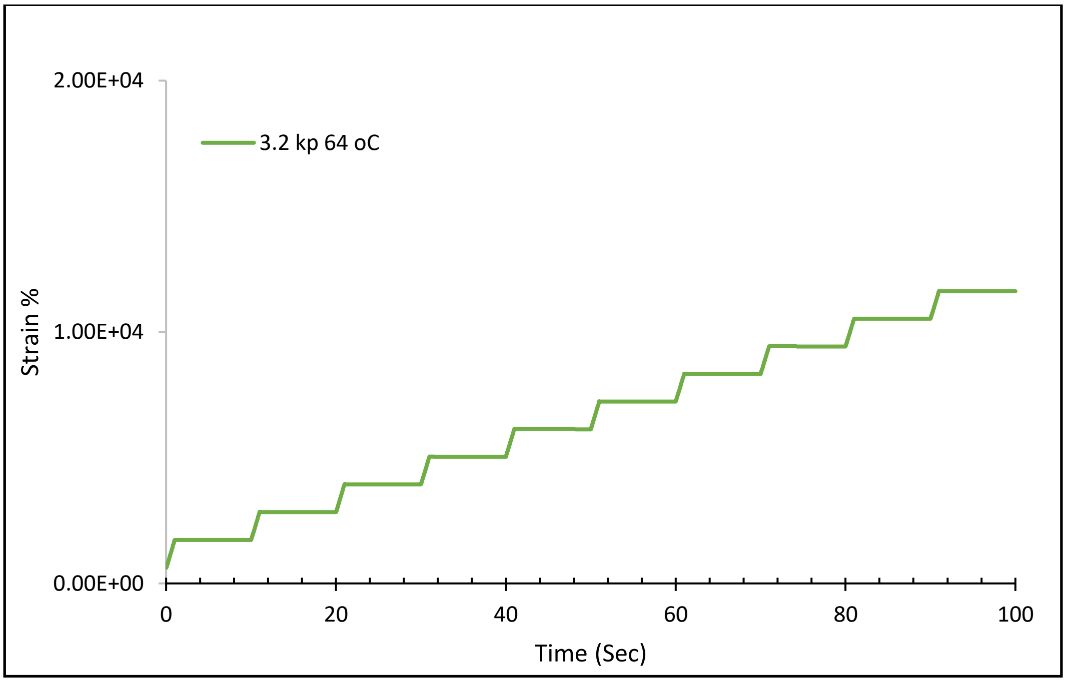


*Figure E-26. 6 % ANSS at 3200 Pa for 64 $^{0}$c*

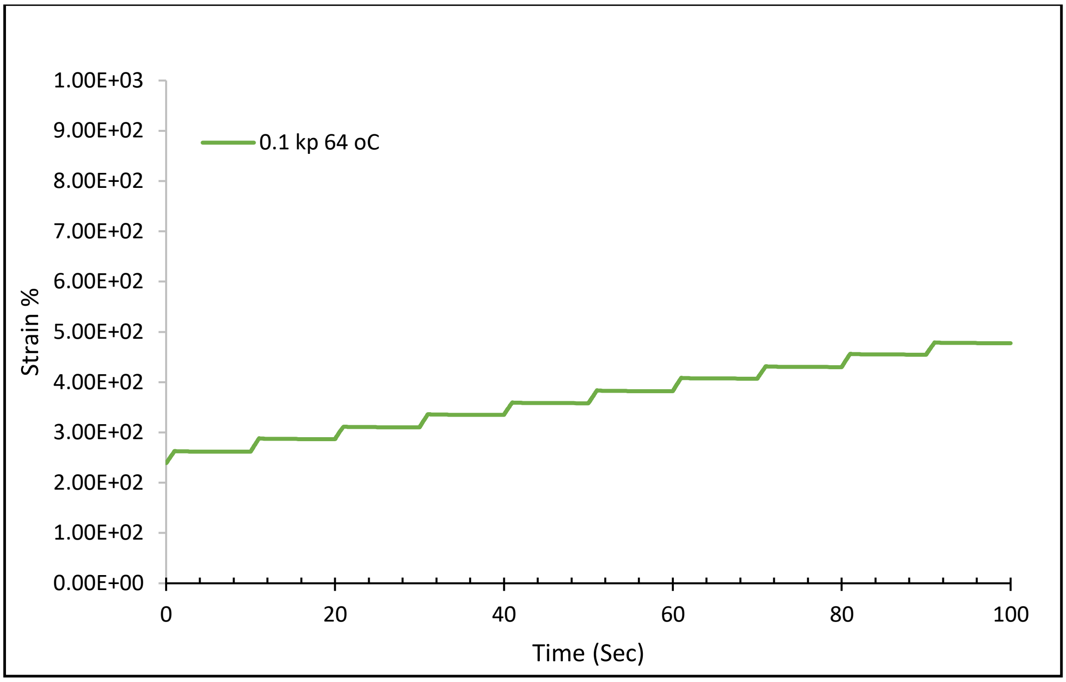


*Figure E-27. 9 % ANSS at 100 Pa for 64 $^{0}$c*

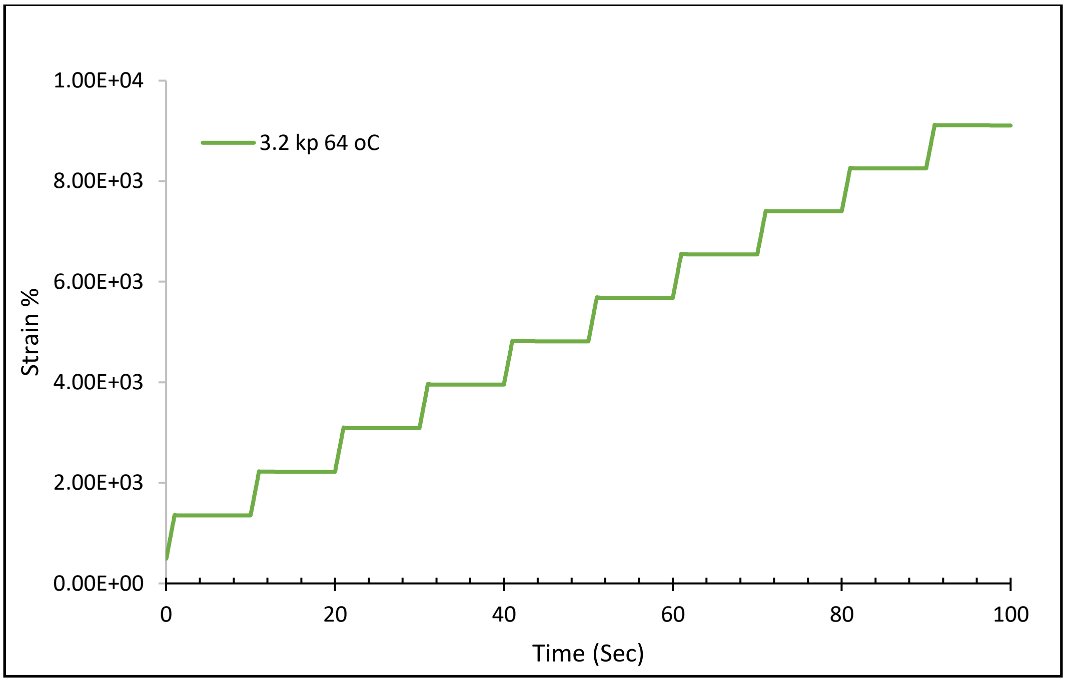


*Figure E-28. 9 % ANSS at 3200 Pa for 64 $^{0}$c*

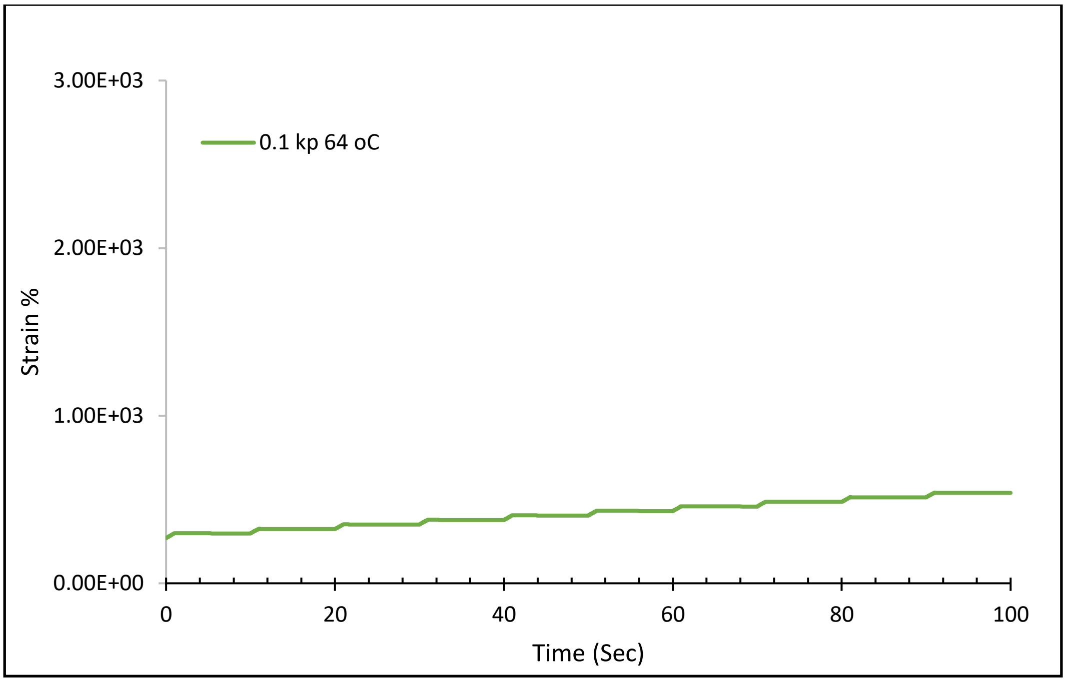


*Figure E-29. 12 % ANSS at 100 Pa for 64 $^{0}$c*

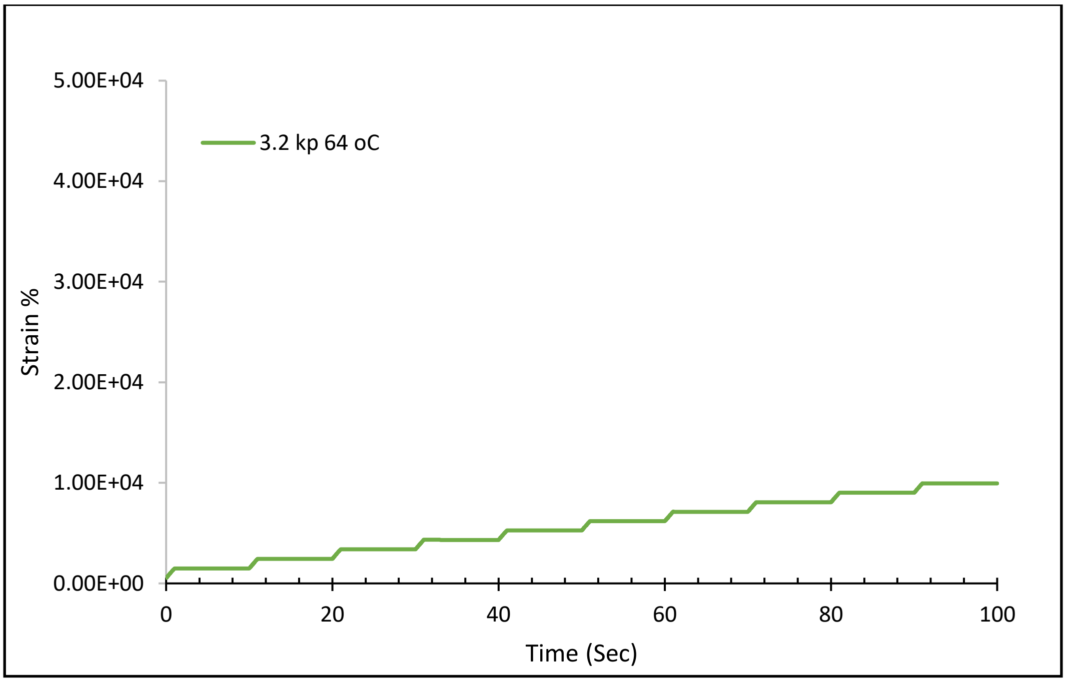


*Figure E-30. 12 % ANSS at 3200 Pa for 64 $^{0}$c*

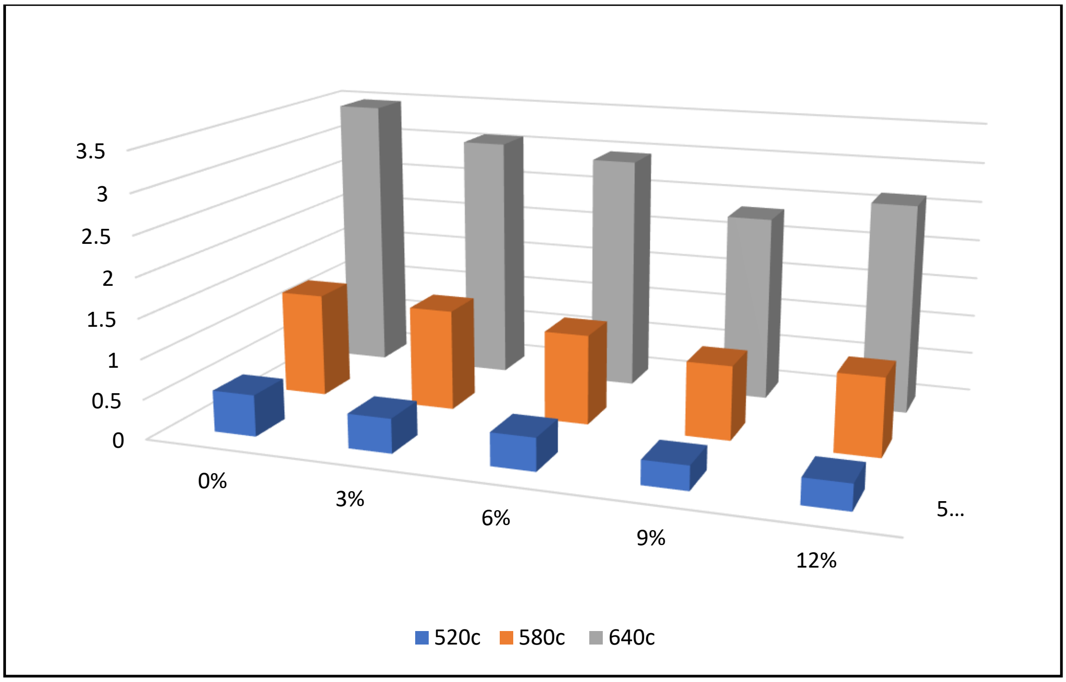


*Figure E 31. The Effect of ANSS % on Jnr at 100Pa Stress*

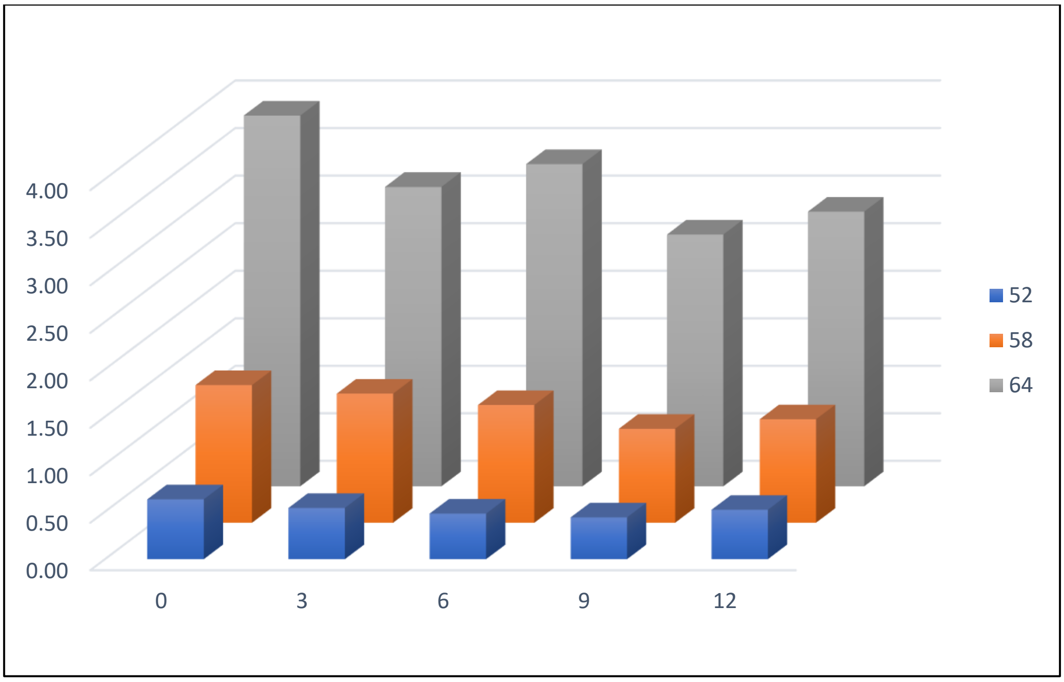


*Figure E 32. The Effect of ANSS % on Jnr at 3200Pa Stress*

# APPENDIX F- SAMPLE PHOTOS


COAST